COMITE EURO-INTERNATIONAL DU BETON

CEB-FIP MODEL CODE 1990

DESIGN CODE

Thomas Telford

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PREFACE

The CEB/FIP Model Code for Concrete Structures was published in 1978 following approval by the Euro-International Committee for Concrete (CEB) at its 19th Plenary Session in Granada in September 1977—the publication was associated with the 8th Congress of the International Federation for Prestressing (FIP) in London in May 1978.

Since that time, the Model Code has had a considerable impact on the National Codes in many countries and, more particularly, on the harmonization of the codification process, as exemplified by the activities of the Commission of the European Communities (CEC), the Eastern Countries, the Nordic Building Regulations Committee (NKB) and members of the European Free Trade Association (EFTA). Indeed Eurocode 2 'Design of Concrete Structures, Part 1: General Rules and Rules for Buildings' used as its basic reference document the Model Code of 1978; this Eurocode was produced under the auspices of the CEC and EFTA members through the Comité Européen de Normalisation (CEN). The CEB activities in its Commissions also contributed to this work.

Naturally the work of the CEB, in synthesizing research findings and technical information with a view to translating them into practice, has continued and, at a certain stage, it became apparent that a revision of the Model Code could, with advantage, be undertaken. Thus the target of establishing the first complete draft of the Model Code 1990 was set and the CEB, together with FIP, worked towards that goal. A Committee for the Model Code (CMC) was set up under the Chairmanship of Professor T.P. Tassios, having an Editorial Board of E. Skettrup, U. Litzner, M. Miehlbradt, J. Perchat, and E. Siviero and with a supporting secretariat, originally in Athens and then in Copenhagen. Membership of the CMC comprised the Chairmen of the various Permanent Commissions and certain General Task Groups of CEB, designated Chairmen of FIP Committees and a number of invited experts.

The first complete draft of the Model Code 1990 was presented for consideration during the 11th Congress of the FIP in Hamburg in June 1990 and for consideration and approval at the 27th Plenary Session of the CEB in Paris in September 1990. Subsequently, the Committee for the Model Code reviewed all the comments received and, acting on the basis of the Technical Resolutions of the 27th Plenary Session, has produced this definitive version of Model Code 1990 for ratification during the 28th Plenary Session of CEB in Vienna in September 1991.

The Model Code 1990, in its drafting, has already influenced the codification work going on concurrently in the National and International fields—a natural effect of the inherent dissemination process occurring within and between international and professional scientific Assocations—and will certainly influence the future codification process, which is a common aim of the CEB and FIP.

On behalf of our two Associations we must thank all those concerned with this work for their sustained efforts to bring the Model Code 1990 to a successful conclusion. These thanks go particularly to those who bore the main burden of the work—the Chairmen and Editorial Board of the CMC: their enthusiasm for, and dedication to, the task were notable.

Finally, both CEB and FIP commend the Model Code 1990 for study and use to all those concerned with, and about, the design and construction of concrete structures which are appropriate to our time and effective, efficient and economic in performance and use.

Roy E. Rowe, President of CEB
René Walther, President of FIP
June 1991

INTRODUCTION

Nature of the Model Code

This document synthesizes scientific and technical developments over the past decade in the safety, analysis and design of concrete structures. It is intended to serve as a basis for the design of buildings and civil engineering works in structural concrete using normal-weight aggregates. Some of the detailed provisions given are only applicable under specified conditions. This Model Code does not attempt to cover particular types of civil engineering works (bridges, reservoirs, off-shore structures) nor does it include provisions against certain actions (seismic, impact or fire), these subjects being treated elsewhere in specific CEB Bulletins and FIP Publications.

By virtue of its international character, this document is more general than most national Codes and, since it is also a Model Code, it provides more detail to aid the drafters of those national Codes in their task of simplifying within their known constraints.

Nevertheless, it is meant to be operational with or without such further simplifications. In this regard the following breakdown of its content will be useful.

- Chapters 1–3 contain basic information serving both as a foundation for the subsequent chapters and as a source of data generally applicable in fields not directly covered by the Model Code. They thus provide an essential data bank for the designer and a basis for further development.
- Chapters 4–10 form the main operational part including specific provisions for the design of concrete structures.
- Chapters 11–14 relate primarily to the construction phase although additional sections on the design of precast structures are also included here.
- The appendices are technical sections associated with the application of this Model Code or for general use.

While this Model Code may be used as a basis for future harmonization of regulatory documents, some of its more innovative contents may, after further calibration, need additional elaboration and this activity will be continuing within CEB, with the outcome being appropriately published.

It is to be noted that the treatment of plain concrete is not fully given.

The format of the Model Code follows the CEB-FIP tradition.

- *On the right-hand side, the text*—the main provisions are presented in the logical sequence of topics. Structural requirements are stated followed by the relevant design criteria, i.e. appropriate engineering models and/or design rules; their application is intended to achieve the satisfaction of the relevant structural requirement. However, the fulfilment of the requirement may alternatively be achieved by means of models other than those given in the Code, provided that the designer is satisfied that adequate substantiating and well founded evidence exists for them in the international literature.
- *On the left-hand side, the comments*—explanations of provisions, particular sketches, alternative simplified rules, indicative numerical values, short justifications of options on the right-hand side, cross references to clauses of this Code or other relevant documents.

  In this connection the Model Code makes use of the following reference documents:

CEB-Bulletin d'Information 124/125 (1978): International System of Unified Standard Codes of Practice in Structures, Volume I: Common Unified Rules for Different Types of Construction and Materials (Joint Committee on Structural Safety–JCSS)
CEB-Bulletin d'Information 191 (1988): General Principles on Reliability for Structures – A commentary on ISO 2394 —approved by the Plenum of the Joint Committee on Structural Safety.

For the application of this Model Code it is assumed that an adequately high quality assurance level is maintained over the design and construction process; thus:

— design is carried out by appropriately qualified and experienced personnel; depending on the importance and complexity of the structure being designed, additional qualifications and/or design checks may be needed;
— construction materials and components are produced and used as specified in the standards and recommendations;
— construction is carried out by appropriately qualified and experienced personnel; depending on the importance and complexity of the structure, the Quality Management System will define required skills and experience of staff, supervision procedures etc;
— the structure, during its intended life, is used as foreseen in the design documents and it is appropriately maintained.

Background
The synthesis of experience and research output and its translation into practical documents for design has been the vocation of the CEB since its establishment; the long tradition of publication of such guidance (Bulletins, Guides, Manuals) periodically culminated in the production of Code-like Recommendations, 1964 and 1970, and, particularly, with the CEB-FIP Model Code 1978.

The impact of MC 78 was very considerable including its direct application, in one form or other, in some twenty-five national and regional Codes. Since 1978, the Model Code has aided the harmonization of the codification process as exemplified by the activities of the Commission of the European Communities (CEC), the Eastern Countries, the Nordic Building Regulations Committee (NKB) and members of the European Free Trade Association (EFTA). Indeed Eurocode 2 'Design of Concrete Structures, Part 1: General Rules and Rules for Buildings' used as its basic reference document the Model Code 1978.

With further developments in the understanding of the performance of concrete and the means of improving analytical and design techniques to reflect that understanding, the continuing work of CEB led naturally to the need to consider revising the existing Model Code. The procedure to be adopted was defined after much preliminary work, and following debates in the CEB Administrative Council and Plenary Sessions, which culminated in the setting up of a 'Committee for the Model Code 1990', with a remit and time scale, under the Chairmanship of Professor T.P. Tassios. This was in May 1987 with the Committee constituted as follows:

J. Appleton, T. Balogh, J. Calavera, J. Eibl, M. Fardis, H. Hilsdorf, M. Kavyrchine, G. König, F. Levi, U. Litzner, G. Macchi, H. Mathieu, M. Miehlbradt, H.R. Müller, J. Perchat, U. Quast, P. Regan, S. Rostam, E. Siviero, E. Skettrup, G. Somerville, G. Thielen, B. Westerberg, M. Wicke, *Ex officio members*: R.E. Rowe,

Y. Saillard, R. Molzahn (until October 1989), R. Tewes (since November 1989).

Experts invited by the Council who participated were

M.A. Chiorino, R. Eligehausen, R. Favre, E. Grasser, P. Matt, M. Menegotto, J. Schlaich, J. Walraven, E. Wölfel.

The work of CMC 90 and its Editorial Board led to the first complete draft of MC 90 (CEB-Bulletins d'Information 195 and 196), which was circulated for comments by the National Delegations and individual members of the CEB. These comments were compiled and, together with the initial reactions of the CMC to them, appeared in Bulletin 198, as an Addendum to the First Draft. All of this material was discussed at the 27th Plenary Session in Paris in 1990.

The Plenary Session approved the Model Code subject to certain agreed modifications and instructed the CMC to finalize the document under the authority of the Administrative Council. All the comments have been carefully considered by the CMC, at its meetings and those of its Editorial Board, and the final text has been achieved on a consensus basis within the Committee.

Thus, after five years of continuous effort, the final version of CEB-FIP Model Code 1990 was published in Bulletins 203, 204 and 205 for ratification at the 28th Plenary Session in Vienna, September 1991.

It is worth noting that with the publication of both Eurocode 2 and the Model Code 1990 at about the same time, the latter will be of special interest and value in commenting on the use of the former during the trial period.

Main innovative aspects

Broader field of application. The CEB-FIP MC 90 has certain characteristics.

a) In its concept and generality, it covers different types of structure as well as buildings. The general content of its chapters has a more fundamental character and is performance orientated.
b) The fundamental character of the contents covers
   i) the description of the mechanical behaviour of reinforced concrete, the materials and their composite behaviour;
   ii) a coherent framework for the subsequent chapters with appropriate simplifications of the basic models;

   and enables designers of exceptional structures, or coping with design situations not covered by this Code, to apply these basic models with confidence but, obviously, with appropriate judgement.

Hence, MC 90 is intended to be operational for normal design situations and structures.

Extensive presentation of properties of concrete. A comprehensive and detailed compilation of the mechanical and other properties of concrete is given. This provides a scientifically sound data base, directly applicable in modern designs, and of general validity.

Generalized behaviour models of reinforced concrete. A series of fundamental behaviour models is included in the Code, as a basis for the dimensioning criteria used in subsequent chapters where, when necessary, further simplifications and adaptations are made.

Broader structural analysis provisions. All types of structural analysis are foreseen. Emphasis is, however, given to linear and non-linear analysis. Additional information is included on the analysis of two-dimensional elements.

Continuity and consistency of models for dimensioning. While the resistance model for axial action effects remains practically the same as in MC 78, a clearly more rational approach is adopted for dimensioning.

Instead of critical 'cross-sections' and separate verifications for axial (M, N) and shear (V, T) action effects, the concept of critical regions is introduced and their global resistance (against M, N, and V, T) is sought. To this end, continuous fields of compressive and tensile forces are considered; their direct effects on the extreme chords and the web of the building elements are examined. Thus shear resistance modelling becomes more rational, and the M, N or M/V and T 'interactions' have a better treatment.

'Discontinuity' critical regions or entire two-dimensional structural elements (plates), where plane sections do not remain plane, are treated in a rational way.

On buckling, several modifications and improvements are introduced such as the 10% criterion, the slenderness bounds, the definition of eccentricity, the simplified calculation of second order effects, and the lateral buckling of beams.

Computational verification against fatigue. Several possible levels of the treatment of the ULS of fatigue are included.

Improvements in Serviceability Limit States verifications. Crack-width control remains as a criterion. In general, crack width limitation is achieved by appropriate detailing based upon stress control. Calculations are only required for special conditions of sensitive steels and corrosive environments; improved modelling is used for this purpose and extended to prestressed concrete. Control of deflections is achieved using methods consistent with the application of the basic model of previous chapters. However, simplified rules are given for most practical cases.

Broader handling of prestressed concrete. Compared with Model Code 1978, the present Code contains more information on prestressed concrete and presented in a more systematic way.

Design for durability. A separate chapter is given on the design of durable concrete structures, reflecting the very considerable emphasis that must be placed on this aspect by designers and, indeed, all those associated with the creation and use of structures.

Design of precast structures. Extensive provisions, both conceptual and computational, for the design of precast structures are given together with certain construction requirements.

Construction aspects. There is a systematic consideration of the construction stage and its interaction with the design stage. More detailed recommendations on the execution of concrete works are now included.

Design by testing. Special attention has been given to this aspect of design and operational provisions are included in an appendix.

Position of the Model Code

The position of the Model Code in the broad spectrum of national and international regulatory documents should be clearly understood.

It is a developing 'model' intended for the widest possible use in the design and construction of concrete structures directly by designers for individual structures or, more often, as a guide to those responsible for drafting the various national and international Codes. As such, it goes into considerable detail, addresses a wider audience with many different traditions and technical practices and thus, necessarily, adopts a more theoretical approach to achieve the optimum clarity for ease of understanding. Also, to this end, it offers a detailed Commentary and several Appendices.

Thus MC 90 should provide a sound background, or source document, with which to refine, improve and, possibly, extend the existing national and international documents.

Future action
Following the practice adopted with MC 78, it is intended to carry out a series of trial calculations for a limited range of structures to show applications of the Model Code; these should be of particular value in comparison with the results obtained using other documents. Equally, in time, Manuals and Bulletins on specific aspects will be produced since the vocation of the CEB leads naturally to these.

R.E. Rowe, President of CEB

T.P. Tassios, Chairman of CMC 90

June 1991

CONTRIBUTORS

The various technical committees of CEB and FIP have contributed over the years to the input to the Model Code. A complete list of those who contributed in the various Commissions and Task Groups was given in CEB-News 81. The constitution of Commissions and Task Groups having a direct input from 1987 to the time of the production of MC 90 was as follows

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CONTENTS

PART I DESIGN INPUT DATA

1. BASIS OF DESIGN

1.1. REQUIREMENTS AND CRITERIA

1.1.1. General requirements

Structures should with appropriate degrees of reliability, during their construction and whole intended lifetime, perform adequately and more particularly

- withstand all actions and environmental influences, liable to occur
- withstand accidental circumstances without damage disproportionate to the original events (this is called the insensitivity requirement).

Their intended lifetime should be specified by the client.

Particular requirements liable to be codified in other guidance documents supplementing this Model Code may refer to

- the particular kind of the structure; an example is a requirement on tightness of a tank
- some hazards; an example is paraseismic requirements; more generally dynamic not quasi-static actions should be covered by such documents
- some particular techniques used for the structure; examples are segmental construction and construction with launching.

General and particular requirements may be supplemented by other requirements specified by the authority, e.g. for the impact of the structure or of its erection on the environment, and by the client, e.g. for economy or for aesthetic aspects.

1.1.2. Criteria

The criteria of compliance with the requirements comprise two categories of measures:

- appropriate design procedures, including measures to facilitate inspection and maintenance of vital structural elements during the lifetime of the structure
- quality assurance measures intended to prevent and eliminate human errors.

The intended lifetime is directly considered for measures relating to durability. For the main dimensioning and for reliability verifications, lifetimes are for practical purposes substituted by reference periods t_R.

In the design procedures, various design situations should be identified as relevant, by distinguishing

- persistent situations
- transient situations
- accidental situations.

The various types of design situations are defined by clause 3.2.2 of CEB Bulletin 191.

Generally a reference period of 50 years is adopted for persistent situations, 1 year for transient situations, and accidental situations are considered to be instantaneous.

In many cases judgement is necessary to supplement codified provisions, in order to identify those design situations that are to be taken into account for a particular structure.

The insensitivity requirement is defined in section 2.1 of CEB Bulletin 191. Clause 3.2.3 of this Bulletin gives some guidance on the choice of a design procedure appropriate to limit damages liable to result from identified or unidentified hazards.

1.1.3. Degrees of reliability

In principle a degree of reliability should correspond to a statistically determined rate of failures with regard to the whole set of requirements (for a set of similar structures under similar conditions). For a given structure this rate should be represented by an assessed probability referable to a given period of time. In the present state of knowledge, degrees of reliability can only correspond to individual verifications and can be associated with nominal probabilities which do not account for gross errors; therefore, they do not represent any actual rate of failures.

The degree of reliability is a function of the design procedures (models and values of actions included) and quality assurance measures associated with the design and construction process.

The design data given in this Model Code are only partial. To supplement them on actions it is possible to refer either to

- Appendices 2 and 3 to Volume 1 of Model 78, or
- to regional or national codes.

It is at present commonly agreed that for the intended degrees of reliability accepted in Western Europe, the 200 years mean return period of characteristic variable actions (envisaged in the Appendices of CEB Bulletin 191) should be reduced to approximately 100 years for the most common climatic actions, in conjunction with this Model Code.

By acting on design and/or quality assurance, degrees of reliability can be indirectly differentiated to some extent, on the basis of criteria related either to the construction works or to the structural member (e.g. because the expected consequences of a failure would be unequal), or to the causes of a possible failure (e.g. a degree of reliability with regard to earthquake or to permanent loads cannot be the same).

In some cases only reliability differentiation measures are defined in this Model Code (see subsection 1.6.1, which is limited to reliability differentiation by the design). Quality assurance measures are still little codified at the international level, but very significant complementary differentiations already result from different quality assurance measures (commonly more or less supplemented or substituted by common practices), adopted at the national level and possibly distinguished depending on the type of building or civil engineering works. For more details, see CEB Bulletin 202.

1.1.4. Quality assurance measures

In order that the properties of the completed structure be consistent with the requirements and the assumptions made during the planning and the design, adequate quality assurance measures shall be taken.

The philosophy of quality assurance is presented in section 2.5 of CEB Bulletin 191. Fundamentally quality assurance implies 'thinking in advance' for the whole building process.

The possibilities to codify quality assurance are limited. However ISO 8402 and 9001 to 9004 define terminologies and concepts and provide guidelines intended for a series of contractual situations. In construction activities various contractual situations between clients, contractors, subcontractors and suppliers are met; the choice of the relevant standard should be made accordingly.

These standards are essentially intended for industrial mass production; some amendments are necessary to use them for construction activities.

Quality assurance measures are both technical and organizational. Some common cases excepted, they should be specified in a general Quality Assurance Plan, which shall identify the key elements necessary to provide fitness of the structure and the means by which they are to be provided and measured, with the overall purpose to provide confidence that the realized project will work satisfactorily in service—fulfilling intended needs.

Each party involved in the realization of a project should establish and implement a quality assurance plan for its participation in the project. Suppliers' and subcontractors' activities shall be covered. The individual Quality Assurance Plans shall fit into the general Quality Assurance Plan.

A Quality Assurance Plan shall define the tasks and responsibilities of all persons involved, adequate control and checking procedures and the organization and filing of an adequate documentation of the building process and its results.

The extent of Quality Assurance Plans should depend on the type and size of the project. In common cases, where current good practice of participants is enough to assure quality, specific Quality Assurance Plans may be unnecessary.

The Quality Assurance Plans and quality controls are dealt with in chapter 12 of this Model Code. The assessments of aspects pertaining to human behaviour (motivation of participants and possibly their skill, qualification, etc.) are widely subjective and cannot be codified. For guidance, refer to CEB Bulletins 157 and 184.

Checklists may be a useful tool for establishing and implementing a Quality Assurance Plan. See examples in CEB Bulletin 184.

1.1.5. Design procedures

Design procedures consist of

- calculations based on analytical models and possibly on experimental models
- applying complementary rules dealing with detailing, limit measures and special provisions.

Detailing, limit measures and special provisions supplement the use of models for various purposes

- to avoid superfluous calculations, e.g. for alternate moment areas
- to satisfy the insensitivity requirement (see sections 2.1 and 3.2.3 of CEB Bulletin 191) with regard to unidentified or hardly quantified hazards; examples are minimum resistance to lateral forces, multiple load paths and ties between structural elements
- to ensure the validity of calculation models, e.g. by minimum ratios of reinforcement
- to ensure good execution and/or durability, e.g. by maximum ratio of reinforcement and by rules for concrete cover.

1.2. LIMIT STATES

1.2.1. General

Limit states are states of the structure defining unfitness for use. They are generally classified as

- ultimate limit states
- serviceability limit states.

Some of them, in turn, may be further subdivided as mentioned in the following clauses.

The limit states either refer to the entire structure, to structural elements or to local regions of elements.

This is only a simplified classification. In very particular cases some intermediate limit states should be considered. For more details, see section 3.1 of CEB Bulletin 191.

1.2.2. Ultimate limit states

Attainment of the bearing capacity of a structural part or of the structure as a whole is classified as an ultimate limit state.

The ultimate limit states considered in this Code are distinguished by the consequences of their exceedance

- loss of static equilibrium (see subsection 1.6.5) when a structural part or the whole structure (considered as rigid bodies) is overturned, is lifted or slides
- exceedance of resistance of one or more critical regions of the structure.

The criteria for resistance refer to the structural behaviour of one or several isolated critical regions of the structure. Under certain conditions, the behaviour of a set of critical regions is considered globally, as a transformation of the structure into a mechanism (see clause 1.6.2.2).

In practice each criterion generally refers to damaging events which occur once only (see clause 5.4.1.4) and result (see subsections 1.6.2 and 6.3.2 to 6.3.5) in a combination of

- axial action effects (bending moments and axial forces)
- tangential action effects (shear forces, torsional moments), including effects on bond and anchorage (section 6.9)

or, in some cases, in each of them separately.

Where second-order effects are to be taken into account in a simple way, it is customary to consider the specific ULS of buckling (sections 1.6.3 and 6.6). Under essentially repetitive loading, the particular ULS of fatigue (sections 1.6.4 and 6.7) is considered.

1.2.3. Serviceability limit states

The serviceability limit states associated with the general requirements refer to

- limited local structural damage such as excessive cracking or excessive compressive stresses, producing irreversible strains and microcracks; see sections 1.6.6, 7.3 and 7.4
- deformations which produce unacceptable damage in non-structural elements or excessively affect the use or appearance of structural or non-structural elements; see sections 1.6.6 and 7.5
- vibrations resulting in discomfort, alarm or loss of utility; see sections 1.6.6 and 7.6.

Large and repeated exceedance of such limit states may result in ULS; however such an exceedance is usually covered by the other verifications.

Such damage reduces the durability of the structure and may also affect its efficient use (tanks, pipes, canals) or the appearance. In many cases, the risk of damage is indirectly covered by ultimate limit state verifications or by detailing.

The conditions to be fulfilled for deformation verifications are associated with the type of building or of civil engineering works. They are often as a simplification, substituted by rough approximations.

Although such limit states may be characterized by the magnitude of the vibrations, they are commonly indirectly covered by limiting the fundamental

period of vibrations of the structure (or some of its members), in comparison to the expected period of the cause of the vibrations.

Other serviceability limit states may be associated with particular requirements (see subsection 1.1.1).

Rare, frequent and quasi-permanent combinations of actions defined in clause 1.6.6.5 are associated with the relevant limit states.

1.3. DESIGN METHODS

For each relevant limit state verification, a design model should be set up from

- an appropriate description of the structure, of its constitutive materials and of its environment
- corresponding behaviour models for the whole or parts of the structure, related to the relevant limit states
- models describing the actions and how they are imposed.

In some cases, this model may be based on experimental tests made for the particular design or on a combination of testing and analytical calculations (see Appendix c).

Variables taking into account chemical and biological influences on material properties are defined in subsection 1.5.2 as exposure classes.

In the model, variables are represented by design values for the relevant limit state.

For some variables, designated as fundamental basic variables, design values include reliability margins.

For other variables, whose dispersion may be neglected or is covered by a set of partial factors, they are normally taken equal to their most likely values.

These reliability margins, defined in section 1.4, seem to cover the whole set of uncertainties; however, a part of the model uncertainties is commonly directly covered by the codified model itself.

In this Model Code the following are considered as fundamental basic variables

- actions (F), unless otherwise specified in particular clauses

This does not exclude that some actions (e.g. shrinkage) can be negligible in particular cases. What is to be considered as one individual action is defined in the corresponding standard and explained in clause 4.2.1 of Bulletin 191. For prestress, see subsection 1.4.3 of this Model Code.

- some geometrical quantities (a), as listed in clause 1.4.5.1
- strengths (f), unless otherwise specified, and other material properties (e.g. creep and friction coefficients) where specified.

For these fundamental geometrical quantities, tolerances should be carefully fixed (see subsection 1.4.5) and controlled. For the other geometrical quantities, tolerances generally reflect usual practice. For all geometrical quantities it would not be realistic to specify tolerances less than twice the mean deviation expected or minimum attainable. As a consequence, tolerances may, according to the case, be either the basis for the design or necessary complements to the design.

Occasionally other variables should be considered as fundamental variables. This may be the case for the numbers of repetitions of loads in fatigue verifications.

See sections 4.1 and 6.1 of Bulletin 191. To identify and select the other relevant fundamental variables is one of the major responsibilities of a designer who faces a problem having some unusual aspects.

Although limit state equations representing different limit state conditions are various, the corresponding design conditions may often be written in the general form

$$S < R \quad \text{or} \quad s \subset R^* \qquad (1.3\text{–}1)$$

where R^* defines a safe domain in which the vector s should be included; the assessment of S may be referred to as overall analysis, while the assessment of R may be referred to as local analysis.

See clause 3.2.1 of Bulletin 191 and the various clauses of section 1.6.

According to the limit state under consideration, the limit state equations may have to be formulated

- either in the space of internal and external moments and forces and directly presented as in eq. (1.3–1), or
- in the space of forces, as

$$F_S < F_R \qquad (1.3\text{–}2)$$

(F_R being for example a carrying capacity), or
- in the space of stresses as

$$\sigma < \alpha f \qquad (1.3\text{–}3)$$

- or in the space of geometrical quantities, as

$$e < C \qquad (1.3\text{–}4)$$

(C being for example a deflection or a crack width).

1.4. METHOD OF PARTIAL FACTORS

1.4.1. General

The partial factor format separates the treatment of uncertainties and variabilities originating from various causes. In the verification procedure defined in this Model Code the design values of the fundamental basic variables (defined in section 1.3) are expressed as follows.

This separation is theoretically not correct, and in practice not complete, because the various factors are not mutually independent. Hence, constant values given in partial factors should be considered as approximations having limited fields of validity. This approximation of using constant values for partial factors may not apply in the following cases

- non-linear limit state equations
- mutually correlated variables
- design by testing.

(a) Design values of actions are generally expressed as

$$F_d = \gamma_F F_{rep}$$

where

F_{rep} are representative values of actions, defined in subsection 1.4.2

γ_F are partial safety factors.

However some actions (e.g. non-closely bounded hydraulic actions) should be expressed in another way, as mentioned in section 4.1 of Bulletin 191. Furthermore for verifications relating to fatigue and vibrations, the format is generally different (see clauses 1.6.4. and 1.6.6.2(c)).

As explained in sections 6.2 and 6.6 of Bulletin 191, γ_F may in some cases be substituted by two partial factors: γ_{Sd} applicable to the action-effect and γ_f applicable to F_{rep}.

For practical applications see clause 1.6.2.4

(b) Design values of strengths are generally directly expressed as

$$f_d = f_k/\gamma_M$$

where

f_k are characteristic values of strengths, defined in subsection 1.4.4

γ_M are partial safety factors.

For material properties other than strengths (e.g. modulus of elasticity, creep, friction coefficients) see the relevant parts of chapters 2, 3 and 4.

Numerical values of γ_M may be different in various parts of the limit state equation, especially for the calculations of S and R; for example (see clause 1.6.2.4(b)) γ_M may be reduced for the assessment of S by a non-linear analysis.

For concrete and steel, γ_M usually covers the deviations of structural dimensions not considered as fundamental variables and includes a conversion factor η converting the strength obtained from test specimens to the strength in the actual structure. For practical application, see clause 1.6.2.4(b).

Other factors, applied to f_d or implicitly included in design formulae, take into account the variations of strength due to non-standardized loading conditions. For practical application, see section 3.5.

As explained in sections 6.3 and 6.6 of Bulletin 191, γ_M may in some cases be substituted by

- one or two partial factors γ_{Rd} applicable to the resistance,
- and a partial factor γ_m applicable to f_k.

(c) Design values of geometrical quantities to be considered as fundamental basic variables are generally directly expressed by their design values a_d.

Liquid levels representing hydraulic actions should in some cases be expressed as $a_k + \Delta a$, where a_k is a characteristic level and Δa an additive or reducing reliability margin.

1.4.2. Representation of actions

1.4.2.1. Definitions and classifications

Actions should be classified as

- direct or indirect
- permanent, variable or accidental
- fixed or free
- static, quasi-static or dynamic
- closely bounded or non-closely bounded.

For practical classifications of the most common actions, see the relevant Appendices to ISO 2394 and Bulletin 191.

Permanent actions, selfweight included, although usually classified as fixed, may have to be considered as partially free where the effects are very sensitive to their variation in space, e.g. for static equilibrium and analogous verifications (see subsection 1.6.5).

Dynamic (not quasi-static) actions are not dealt with in this chapter.

Reactions, mainly on supports, should also be distinguished from directly imposed actions. Although they are taken into account like actions for some verifications, they are in reality effects of actions and may need specific reliability measures in design.

Soil reactions, e.g. soil pressure underneath foundation slabs or footings, are strongly influenced by soil-structure interaction. They should be determined by analysis; but the result should commonly be considered widely uncertain, especially the distribution in space.

1.4.2.2. Representative values

(a) Permanent actions

Each permanent action is represented by a single representative value G if at least one of the following conditions is satisfied

- the variability of the action in time and with regard to the design is small
- the influence of the action on the total effect of the actions is small
- it is evident that one of the two representative values (the upper or the lower) governs all parts of the structure.

In the first two cases, G is considered as a mean value and should be calculated from nominal dimensions. In the third case it is defined as G_{sup} or G_{inf}.

The difference between G_{sup} or G_{inf} and G_m should not exceed 0.1 G_m. For some types of prestressed structures this maximum acceptable difference may have to be reduced to 0.05 G_m.

In the other cases, two representative values (upper and lower, G_{sup} and G_{inf}) should be defined, taking into account variations which can be foreseen.

Nominal numerical values of densities are given in subsection 2.1.2 for plain, reinforced and prestressed concrete, and in ISO 9194 for other materials. For future possible permanent equipment an upper value should be specified.

The representative values of the prestress are defined in subsection 1.4.3.

This case is mainly applicable to finishes and equipment. G_{sup} and G_{inf} may normally be defined as corresponding to 0.95 and 0.05 fractiles plus (or minus) the expected variation in time of G_m.

(b) Variable actions

Each variable action may be represented by

- characteristic value Q_k
- combination value $\Psi_o Q_k$
- frequent value $\Psi_1 Q_k$
- quasi-permanent value $\Psi_2 Q_k$.

For the most common variable actions these values are given in standards or codes associated with the same γ_F values as in this Model Code. It is also possible to refer to Appendices 2 and 3 to Volume I of Model Code 78 (see 1.1.3 above). Ψ values depend on the model of the action.

In particular cases (e.g. temperature and possibly hydraulic actions) upper and lower values should be distinguished for the three first representative values.

Besides, for some variable actions, specific representative values are defined for fatigue verifications.

These values are associated with the methods of verification defined in clause 1.6.4.2(d).

(c) Accidental actions

Each accidental action can be given by a single representative value, which is usually the design value A_d.

These values are normally defined by the competent public authority or by the client and correspond to the values beyond which a high probability of integrity of the structure can no longer be assured.

1.4.2.3. Load arrangements

For each free action, different load arrangements should be defined.

See subsection 4.2.3 of CEB Bulletin 191.

These load arrangements are sometimes defined in the load standards. If several actions are free, the load cases (fixing the arrangements of all actions by taking into account their compatibility) are sometimes defined in the same documents.

1.4.3. Representation of prestress

Precamber, i.e. a permanent deformation of the concrete structure imposed by jacks during execution, is not treated in this Model Code. Preflexed steel profiles are not covered either.

Stays, i.e. cables in which the tension is mainly due to the permanent weight of the stucture, are not treated in this Model Code.

For more details on the content of this subsection and for its background refer to the report 'Reliability problems associated with uses of prestress' in CEB Bulletin 202.

1.4.3.1. Definition and classification

A tendon or a set of tendons is prestressed if it is subjected to permanent tensile stresses and strains, due to external forces purposely exerted on it.

Generally, during prestressing, the external forces are imposed and the associated elongations of the tendons are controlled.

As defined in the following chapters, the various parts of the calculations (e.g. for the assessment of losses of prestress, for structural analysis and for verifications with regard to various limit states) have to refer to different indicators consistent with these parts of the calculations. These indicators may be, e.g. force or prestrain, with or without the effect of permanent actions. See especially subsection 4.2.1. An example is given in clause 1.4.3.3.

The various types of prestress and anchorages are listed in section 4.1.

In the most common cases the action effects due to prestress can be classified as

- local, i.e. where only the action effects due to prestress are significant
- isostatic (in statically determinate structures)
- isostatic and hyperstatic (in statically indeterminate structures).

Mainly in the vicinity of anchorages and at points where the tendons change their direction.

The distinction between isostatic and hyperstatic effects is not possible for slabs and shells.

1.4.3.2. Representative values of prestress considered as an action

At given time t and abscissa x (or arc length) the prestressing force $P(x, t)$ is equal to the total force $P(0, 0)$ imposed at an active end of the set of tendons, minus losses $\Delta P(x, t)$. Losses can be classified as immediate, i.e. $\Delta P(x, 0)$, and long-term, resulting in final total losses $\Delta P(x, t_R)$ at the end of a reference time t_R. A strain $\varepsilon_p(x, t)$ can be associated with $P(x, t)$.

Even where prestress has to be considered as an action, a prestrain $\varepsilon_p(x, t)$ has commonly also to be considered in some parts of the calculations, especially in verifications with regard to ULS. Where only immediate losses are considered $\varepsilon_p(x, t)$ is deduced from $P(x, t)$ by dividing it by the product $E_p A_p$. Where also long-term losses are considered, this simple division may have to be supplemented by a correction transforming the relaxation of the tendon into a variation of strain.

Length and angular deviation may be considered small if the ratio $\Delta P_m(x, t)/P(0, 0)$ is not, at any time t, greater than 0.30. In other cases see subsection 4.6.2.

They may also depend on whether possible corrective measures are specified in a Quality Assurance Plan.

This general rule, necessary for practical reasons, may result in non-negligible uncertainties on the action effects if several, not correlated, prestresses intervene in the same verification. Besides, attention is drawn, especially for serviceability verifications, to the fact that the variability of the hyperstatic effects is not fully represented by P_k and special care would be necessary in cases where such effects are greater than usual.

In the vast majority of cases, when characteristic values shall be considered, it is sufficient to consider $P_{k\,\mathrm{sup}}$ for the initial persistent situation and $P_{k\,\mathrm{inf}}$ for the final persistent situation.

Losses are numerically defined, as mean values $\Delta P_m(x, t)$, in chapter 4 assuming that the structure is submitted to the quasi-permanent combination of actions defined in clause 1.6.6.5a.

For a given set of tendons, considered in the same calculation of losses, the mean value of the prestressing force is defined as $P_m(x, t) = P(0, 0) - \Delta P_m(x, t)$ (ΔP in absolute value).

Two characteristic values of the prestressing force are also defined.

In the cases where the length and angular deviation of the tendons are not exceptionally large, the following formulae, although conservative if the angular deviation is small, may be used as acceptable approximations.

(*a*) *Bonded tendons*

$$P_{k\,\mathrm{sup}}(x, t) = 1.1\, P_m(x, t)$$

$$P_{k\,\mathrm{inf}}(x, t) = 0.9\, P_m(x, t)$$

(*b*) *Unbonded tendons*

$$P_{k\,\mathrm{sup}}(x, t) = 1.05\, P_m(x, t)$$

$$P_{k\,\mathrm{inf}}(x, t) = 0.95\, P_m(x, t)$$

The representative values of prestressing forces to be used in various verifications are defined in the relevant chapters (4, 6 and 7).

As a general rule, the same representative value P(P_m or $P_{k\,\mathrm{sup}}$ or $P_{k\,\mathrm{inf}}$) is used for all sets of tendons involved in one verification (e.g. a verification with respect to maximum shear at a given abscissa).

In many cases the following design situations shall be distinguished for the structure as a whole

- transient situations during erection, during which losses are calculated for the relevant values of t
- persistent situation (initial situation) defined for prestress by the losses at the beginning of the lifetime of the completed structure (time t_0)
- persistent situation (final situation) defined for prestress by the final total losses.

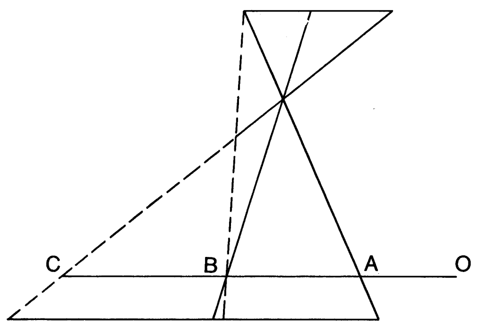
The values of γ_P are specified in the relevant clauses of this Model Code (especially in 1.6.2.4(a). In many cases γ_P is equal to 1.0. In some cases, instead of introducing a γ_P factor different from 1.0, it may be suitable to use an upper limit of the bearing capacity of the tendon.

γ_P mainly covers the variability of the effects of prestress (as the other γ_F do) and the variability of prestress in extreme conditions (i.e. beyond the characteristic values). It does not cover the stress variations due to actions not taken into account in ΔP_m.

For SLS verification γ_P, systematically equal to 1, is not considered.

For calculations with regard to ultimate limit states, the design value of the prestress is deduced from the relevant representative values of $P(x, t)$ by multiplying by a partial factor γ_F, denoted $\gamma_P(\gamma_{P\,\mathrm{sup}}$ or $\gamma_{P\,\mathrm{inf}}$ if relevant).

When a part of a calculation refers to an indicator other than the value of P defined in clause 1.4.3.2, e.g. to the difference of strain between the tendon or the adjacent concrete, the steel stress variations are calculated in two steps: a first part $\Delta\varepsilon_1$ corresponding to the difference between $\varepsilon_p(x, t)$ and the indicator, and the second part $\Delta\varepsilon_2$ corresponding to the rest of the design values of these actions. As an example see Fig. 1.4.1.



B corresponds to the indicator defined as the difference of strain between a tendon and the adjacent concrete (see subsection 6.1.1)

AO represents $\varepsilon_P(x, t)$ (this value is used in clause 4.6.2)

AB represents $\Delta\varepsilon_1$ (before cracking at the level of the tendon)

BC represents $\Delta\varepsilon_2$ (after cracking)

CO represents the design strain used for the assessment of the resistance (see subsection 6.2.4)

Fig. 1.4.1.

1.4.3.3. Stress variations of prestressing steels

Steel stress variations due to actions, the effect of which on $P(x,t)$ is not included in P_m are calculated on the basis of the design values of these actions. No supplementary creep is taken into account in the assessment of these variations.

The design value of the yield stress, to be considered in this case, would be $\gamma_s f_{ptk,\,\sup}$, which in practice never can be exceeded.

If unbonded prestress is used with tendons set within the external outline of the structure, the stress variations $\Delta\sigma_p$ may be neglected. If tendons are partially set out of this external outline, a more refined analysis of deformations is necessary to calculate $\Delta\sigma_p$.

Practically, where the simplified stress–strain diagram defined in Fig. 2.3.2 (clause 2.3.4.3), but transformed to a design diagram by dividing $0.9 f_{ptk}$ by γ_s, is used, the method can be developed as follows.

The stress variations $\Delta\sigma_p$, due to design external actions, are first calculated for every critical cross-section.

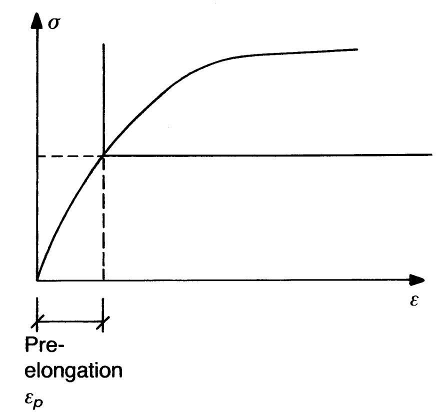
A first verification then consists of checking whether the total stress of all tendons is less than their design yield stress $0.9 f_{ptk}/\gamma_s$. If this condition is satisfied, prestress and stress variations are considered, as in (a), for the rest of the verification for the same critical regions.

This condition should always be satisfied for SLS verifications (with in this case $\gamma_s = 1$). In practice it is covered by the verifications for ULS, or relating to reinforcing steels, considered in this Model Code.

If the condition is not satisfied, the isostatic effect of prestress of these tendons and their stress variations are globally substituted by $0.9 f_{ptk}/\gamma_s$ for the rest of the verification for the same critical region, as if the tendons were considered to be passive.

(a) Where $P_{k\,\sup}$, (or $\gamma_{P\,\sup} P_m$ with $\gamma_{P\,\sup} > 1$) is considered, which implies that an increase of prestress would be considered to be unfavourable, no reference is made to any yield design strength limiting the stress, even if external actions result in stress increases in the tendons. The design prestressing force is then considered as acting, i.e. as an external action, to assess the total action effects applied to the structure, and the stress variations of the tendons are either neglected or taken into account as the stresses of reinforcing steels for the assessment of the structural response.

(b) Where $P_{k\,\inf}$ (or $\gamma_{P\,\inf} P_m$ with $\gamma_{P\,\inf} \not> 1$) is considered, which generally implies that an increase of prestress would be favourable, reference is made to the design stress–strain diagram for the prestressing tendons (with γ_s for ULS verification). The prestressing force is in principle considered as acting. The stress variation $\Delta\sigma_p$ is calculated by shifting the origin of the design stress–strain diagram for the prestressing tendon by an amount corresponding to the pre-elongation ($\varepsilon_{Pk\,\inf}$ or ε_{Pm}) (Fig. 1.4.2) associated with the relevant indicator—and is taken into account separately as is stress of reinforcing steels (generally for the assessment of the resistance of the critical region).



1.4.4. Representative values of material properties

Strengths and other material properties to be considered as fundamental basic variables are represented by their characteristic values f_k (or X_k) or by their mean values.

The significance of these values is shown in clause 6.3 of Bulletin 191. In exceptional cases, where an increase of the strength results in a decrease in reliability, upper characteristic values and specific γ_M values (smaller than 1) should be used.

Some formulae given in section 2.1 make it possible to assess the representative values of some material properties from strengths, with which they are correlated (e.g. f_{ct} and E_c from f_c). These formulae should be used for assessing the mean values and the characteristic values, but not the design values.

Where strengths and other material properties are not considered fundamental variables in limit state equations (see sections 1.3 and 1.6), they should be represented by mean values f_m (or X_m), which usually are the most likely values of f, and not other fractiles taken out of the same statistical populations as f_k values. However, these may generally be substituted by characteristic values f_k, as an approximation acceptable for such verifications.

1.4.5. Geometrical quantities

1.4.5.1. Representative values

Unintentional eccentricities, inclinations and parameters defining curvatures affecting columns and walls and the depth of reinforcement in members thinner than 100 mm, are the unique geometrical quantities defined in this Model Code to be taken into account as fundamental basic variables.

The other geometrical quantities are given the numerical values specified in the drawings of the design.

The fundamental geometrical variables are directly fixed as design values in the chapters where the relevant limit states are treated.

In this clause, only geometrical quantities representing the structure are considered (for geometrical quantities representing some actions, refer to subsections 1.4.2 and 1.4.3). For most of the quantities, their deviations within the specified tolerances should be considered as statistically covered by γ_{Sd} and γ_{Rd}, i.e. by γ_F and γ_M factors. Only those quantities, which might in some verifications be one of the main variables, should, in those verifications only, be taken as fundamental.

The depths of reinforcement in thin members are taken into account by modifying their nominal values by additive reliability margins.

1.4.5.2. Tolerances

The possible deviations in the geometry of the concrete elements, of the cover, or of the position of steel shall not alter significantly the SLS nor the ULS performance of the relevant elements.

As a general rule for these geometrical fundamental variables, the corresponding specified tolerances may be taken equal to their design values of the deviations divided by 1.2 and should be controlled accordingly.

Larger than intended dimensions in slabs may significantly increase the selfweight, whereas smaller dimensions and/or lever arms of steel bars may significantly reduce resistances. Similarly, smaller than nominal values of concrete cover may endanger the durability or the anchorage resistance of steel bars. An unintended inclination of columns may disproportionately increase their action effects.

For the other geometrical variables, the values of the materials partial safety factors included in this Code, are meant to cover small reductions of performance (resistances, mainly) which may result from their deviations.

Because of the complicated nature of the related phenomena, no explicit figure of general validity can be given on the amount of such performance reduction; however, it is considerably less than 4%.

In the absence of a more justified set of tolerances, the following limitations apply.

(a) *Table 1.4.1. Tolerances for concrete sectional dimensions*

Elements and dimension (mm)	Tolerances (mm) $\Delta a = (a_{nom} - a_{act})$
Beams; columns; walls	
$a \leqslant 200$	$\lvert\Delta a\rvert < 5$
$200 < a \leqslant 2000$	$\lvert\Delta a\rvert < 3.5 + 0.008\,a$
$2000 < a$	$\lvert\Delta a\rvert < 17.5 + 0.001\,a$
Slabs	
$a \leqslant 200$	$-10 < \Delta a < 6$
$200 < a \leqslant 2000$	$-20 < \Delta a < 4 + 0.010\,a$
$2000 < a$	$-30 < \Delta a < 20 + 0.002\,a$

(b) *Table 1.4.2. Tolerances for the position of passive or active reinforcement*

Structural depth d (mm)	Tolerances (mm) $\Delta d = (d_{nom} - d_{act})$
$d < 1000$	$\Delta d < 10$
$1000 < d < 2000$	$\Delta d < 0.01\,d$
$2000 < d$	$\Delta d < 20$

(c) Tolerance of cover: $c_{nom} - c_{act} < 10$ mm (see 8.4.3(c)).

(d) Tolerance of unintentional inclination of columns and walls: $\alpha < 1\%$.

The above tolerance values are valid for the final condition of the structure, after compaction and hardening of the concrete.

Depending on the quality assurance scheme applicable, relevant tolerance values should be respected for each category of possible deviations under well specified conditions of measurements and evaluations. Possible higher deviations foreseen, should lead to additional design steps taking into account all the consequences of deviations in exceedance of the specified tolerances.

If a structure is designed, executed and maintained according to the requirements of the Model Code, there is a high probability that it will withstand the expected conditions of use for a long period of time, say 50 years or more.

1.5. PRINCIPLES OF DESIGN VS. DURABILITY

1.5.1. General

Concrete structures shall be designed, constructed and operated in such a way that, under the expected environmental influences, they maintain their safety, serviceability and acceptable appearance during an explicit or implicit period of time, without requiring unforeseen high costs for main-

Designing for durability implies a slow-down of the process of deterioration of the parts of the structure that are critical in this respect. This normally implies a multistage strategy, which may often be based on successive protective barriers against corrosion.

The service life concept leads to an integral treatment of the

- planning and design phase
- execution phase
- period of use.

However, it is not the intention of this Code to impose any legal obligations to third parties, but only to make clear the whole working environment in which the designer is acting.

1.5.2. Exposure classes

Environmental conditions mean those chemical and physical actions to which the concrete is exposed and which result in effects that are not considered as loads or action effects in structural design. In the absence of a more specific study, these environmental conditions may be classified in the exposure classes given in Tables 1.5.1 and 1.5.2.

Table 1.5.1. Exposure classes related to environmental conditions

Exposure class	Environmental conditions
1. Dry environment	E.g. interior of buildings for normal habitation or offices*
2. Humid environment (a) Without frost	E.g. • interior of buildings where humidity is high† • exterior components • components in non-aggressive soil and/or water
(b) With frost	E.g. • exterior components exposed to frost • components in non-aggressive soil and/or water and exposed to frost • interior components when the humidity is high and exposed to frost
3. Humid environment with frost and de-icing agents	E.g. interior and exterior components exposed to frost and de-icing agents
4. Sea-water environment (a) Without frost	E.g. • components partially immersed in sea-water or in the splash zone • components in saturated salt air (coastal area)
(b) With frost	E.g. • components partially immersed in sea-water or in the splash zone and exposed to frost • components in saturated salt air and exposed to frost
5. Aggressive chemical environment‡ (a)	E.g. • slightly aggressive chemical environment (gas, liquid or solid) • aggressive industrial atmosphere
(b)	Moderately aggressive chemical environment (gas, liquid or solid)

* This exposure class is valid only as long as during construction the structure or some of its components is not exposed to more severe conditions over a period of several months.
† E.g. in commercial laundries.
‡ May occur alone or in combination with classes 1–4.

For more information, see d.6.3.

The treatment of corrosion of the reinforcement itself may require additional restrictions.

Table 1.5.2. Limiting values of deleterious substances in water of predominantly natural composition for the assessment of the severity of chemical attack

	Degree of severity		
	Slight	Moderate	High
pH-value	6.5–5.5	5.5–4.5	< 4.5
Carbonic acid dissolving lime $(CO)_2$ in mg/l determined by marble test according to Heyer	15–40	40–100	> 100
Ammonium (NH_4^+)	15–30	30–60	> 60
Magnesium (Mg^{2+}) in mg/l	300–1000	1000–3000	> 3000
Sulphate (SO_4^{2-}) in mg/l	200–600	600–3000	> 3000

For more information, see d.6.6.4.

1.5.3. Durability design criteria

In order to satisfy the requirement of subsection 1.5.1, a combination of criteria may be used as defined in this Model Code and developed in chapter 8 (including structural form, skin-concrete quality, detailing), as well as in chapter 7 for nominal crack width control under specified conditions. Additional protective coatings may be envisaged under very specific conditions.

1.6. BASIC DESIGN RULES

1.6.1. General

The basic design rules applicable to the various types of limit states listed in section 1.2 are presented in subsections 1.6.2 to 1.6.6.

The numerical values of γ factors given in these clauses are applicable to buildings and civil engineering works not subject to variable actions having an exceptional variability.

These basic design rules differ according to the limit state under consideration.
These numerical values are considered as appropriate for the socio-economic conditions in the Western European countries. In some countries (and possibly depending on the type of building or civil engineering works) where different conditions prevail, γ factors may be reduced.

However, the $\gamma_{G\,sup}$ and γ_Q values given in subsection 1.6.2 may be reduced respectively to 1.2 and 1.35 for reliability differentiation in the following cases:

See clause 1.1.3 concerning reliability degrees and reliability differentiation measures.

one-storey buildings (ground floor plus roof) with spans not exceeding 9 m, that are only occasionally occupied (storage buildings, sheds, green-houses, small silos and buildings for agricultural purposes), floors resting directly on the ground, light partition walls, lintels, sheeting and ordinary lighting masts, provided that these reductions are not associated with a reduced quality assurance level.

If the basic set of γ factors given in this section is adopted, any increase of the reliability degree is normally limited to the consideration of supplementary hazards or higher values of accidental actions, and more refined analyses.

Some γ_M factors may however have to be increased in cases where quality measures, considered normal in the actual case, would not be expected, but this is intended to maintain the reliability degree, not to modify it.

In principle all relevant limit states should be considered, as well as all relevant design situations, load arrangements and load cases and combinations of actions.

In some cases, defined in other chapters, some limit state calculations may be substituted by detailing rules or special provisions.

1.6.2. ULS of resistance of critical regions

1.6.2.1. Definition

These limit states have been defined in 1.2.2.

1.6.2.2. Design principle

It should be verified that either condition (a) is satisfied or conditions (b) and (c) are satisfied.

(a) In any cross-section, chord, strut or tie

$S_d < R_d$ if a one-component action-effect is to be considered,
$S_d \subset R_d^*$ if a multi-component action-effect is to be considered,

where

S_d denotes a design action-effect,
R_d denotes a design resistance (and R_d^* a design resistance domain).

(*a*): for example normal force and bending, or shear and torsion.

Models of resistance used in such verifications are denoted 'first yield' models in clause 5.4.1.2 and in chapter 6.

OR

(b) The number and extent of plastic hinges is such that no equilibrium would be possible in the model if the actions were increased.

Such a model can be physically envisaged only if elastic-plastic or rigid-plastic behaviour is assured for the reinforcement.

(*b*): only in some cases in which a statically indeterminate structure is considered.

This is not the only physical condition (see chapter 5). For example such a limit state cannot be reached at the fixed end of a cantilever slab eccentrically loaded. In any case this model is usable only for verifications with regard to bending; tangential limit states (shear, punching) are verified according to (*a*).

Unless the location of the plastic hinges be unconditionally codified, it should be verified that this location is the most unfavourable.
Hence the verification is made by comparing global design loads to maximum proportional loads compatible with design resisting moments.

In this case all indirect actions are neglected. The location of the plastic hinges is identified. Then the design resistances of the plastic hinges are calculated and it shall be verified that a stable equilibrium is statically possible without exceeding these resistances if the loads take their design values

AND

The ULS may be reached before the formation of all plastic hinges.

(c) Using the same models as in (b) (see clause 5.4.1.2), the limit plastic rotation θ_{pl} is not exceeded at any critical section before the mechanism of plastic hinges is fully developed.

These may be chosen separately if their compatibility is sufficiently ensured.
The various types and models may result in safety degrees. These differences should be limited either by corrective factors or (in most cases) by limiting the fields of application of some types and models.

1.6.2.3. Types and models for overall and local analysis

These are defined

- in chapter 5 for overall analysis (of the whole structure)
- in chapter 6 for local analysis (e.g. of cross-sections).

In case (b) of clause 1.6.2.2, only plastic analysis can be used; hyperstatic effects of prestress (see clause 1.4.3.1) are ignored.

1.6.2.4. Partial factors and ways to introduce them into the calculations

The general content of γ_F factors is defined in subsection 6.2.2 of Bulletin 191.

(a) γ_F factors

An example of particular actions is that of some hydraulic actions (see Bulletin 201).

a1. Persistent and transient situations. The numerical values applicable to non-particular actions are given in the following table and clauses.

Basic values given in Table 1.6.1 are in some cases conservative. See subsections 1.1.3 and 1.6.1 (and Bulletin 202) for reliability differentiation and clause 1.6.2.5(c) for possible refinements (especially for $\gamma_{G\,sup}$).
If, as envisaged in subsection 1.4.1 and in (*c*) later, each of these factors is substituted by two partial factors γ_{Sd} and γ_f (respectively γ_g, γ_p and γ_q), the following approximations usually may be accepted.

- γ_{Sd} should generally be assumed to be equal to 1.15 for permanent action and 1.10 for variable actions; taking $\gamma_{Sd} = 1.125$ for both is generally acceptable.
- γ_f (i.e. γ_g, γ_p or γ_q) $= \dfrac{\gamma_F \text{ (i.e. } \gamma_G, \gamma_P \text{ or } \gamma_Q)}{\gamma_{Sd}}$

Table 1.6.1. Partial γ_F factors: basic values

Actions, γ_F	Unfavourable effect (γ_{sup})	Favourable effect (γ_{inf})
Permanent, γ_G (P excluded)	1.35	1.0
Prestress, γ_P	1.1	1.0
Variable, γ_Q	1.5	Usually neglected

This rule is not applicable for the limit state of equilibrium.

In the most common cases one of $\gamma_G(\gamma_{G\,\text{sup}}$ or $\gamma_{G\,\text{inf}})$ may be applied globally to all permanent actions (unfavourable or not), prestress excepted. The other cases should be identified by judgement.

A more refined method is defined in CEB Bulletin 128 par. 9.433.

Action-effects sensitive to random spatial variation of actions (permanent or variable), usually classified as fixed (see clause 1.4.2.1), should be calculated with the assumption that a part ξ of each of these actions is considered a free action. ξ should be determined on the basis of a study of the spatial variability of the actions. If no such study is made, $\xi = 0.1$ may be used.

The reduction of γ_F is for example applicable to a favourable normal force, independent of or little correlated with the unfavourable bending moment. The rule is equivalent to the application of a factor γ_{Sd} equal to 0.95 instead of 1.12 to this component. It is conservative in cases where the components are partially correlated and should ensure against the premature cut-off of bars in columns of a multistorey frame in which the assessment of the favourable normal forces cannot be precise.

For bi-component action effects, if a component is favourable, the γ_F factors associated with this component should be divided by 1.2 if the two components are not correlated or little correlated. In other cases (e.g. for the normal force and the moment due to prestress) the same γ_F factors shall be applied to both components.

For imposed deformations (permanent or variable), γ_F values given in Table 1.6.1 are applicable in the case (a) of 1.6.2.2 if the deformations themselves, and not corresponding forces, are introduced in the structural analysis. Commonly, depending on their origin or effect, imposed deformations may not be taken into account for the ultimate limit state. If they are, in the case of linear analysis with or without redistribution, the partial factors γ_F applicable to them should be between 1 and 1.2.

Unless differently specified in particular clauses, the conditions of use of γ_P are the following.

- The upper value of γ_P (i.e. $\gamma_{P\,\text{sup}}$) given in Table 1.6.1 shall be associated with the characteristic value $P_{k\,\text{sup}}$ and is applicable for general effects (i.e. where the effects of other actions have the same order of magnitude as the effects of P). It shall be increased up to 1.3 for local effects. If for simplification $P_{k\,\text{sup}}$ is substituted by P_m, $\gamma_{P\,\text{sup}}$ shall be increased by 10% if prestress is bonded, by 5% if unbonded.
- The lower value of γ_P (i.e. $\gamma_{P\,\text{inf}}$) given in Table 1.6.1 is normally applicable to $P_{k\,\text{inf}}$. It may generally be kept if applied to P_m, especially for verifications with regard to bending; however, in some cases, especially for some verifications with regard to shear, the more safe-sided value 0.9 may be preferred if applied to P_m.

Generally the same γ_P should be applied to all prestress involved in the same verification, and it is sufficient to associate $\gamma_{P\,\text{inf}}$, applied to $P_{k\,\text{inf}}$ or P_m, with $\gamma_{G\,\text{sup}}$, and to associate $\gamma_{P\,\text{sup}}$, applied to $P_{k\,\text{sup}}$ or P_m, with $\gamma_{G\,\text{inf}}$.

For closely bounded variable actions or low variability actions identified in standards the value 1.5 of γ_Q (unfavourable action-effect) should be reduced to 1.35.

a2. Accidental situations. The values of γ_F applicable to all actions are equal to 1.

Safety is normally ensured by the design values of the accidental action or of the other parameters describing the accidental situation.

(b) γ_M factors

The numerical values of γ_M to be used for calculating R_d are given in Table 1.6.2.

The general content of γ_M factors is defined in subsection 6.3.2 of Bulletin 191.

As a simplification a conversion factor η is included in γ_c.

Table 1.6.2. Partial factors-γ_M

Fundamental basic variable	Design situation	
	Persistent/transient	Accidental
Concrete		
Compressive strength (f_{cck}), γ_c	1.5	1.2
Tensile strength (f_{ctk}), γ_{ct}	*	*
Reinforcing or prestressing steel		
Tensile strength (f_{stk}), γ_s	1.15	1.0
Compressive strength (f_{sck}), γ_{sc}	1.15	1.0

* See relevant clauses.

The values of γ_c and γ_s given in Table 1.6.2 should be increased if the geometrical tolerances given in clause 1.4.5.2 are not fulfilled. Conversely they might be reduced by 0.1 and 0.05 respectively, at the maximum, if these tolerances are reduced by 50% and are strictly controlled (see e.g. subsection 14.1.3).

A variation of γ_c or γ_s, according to the degree of control of f_{ck} (without making the specimen more severe), does not seem to be justified, because the variation of the control can more rationally be taken into account by the compliance criteria included in the control itself. In any case, it cannot be numerically fixed independently of the control criteria. Besides, even if a better quality, characterized by a lower coefficient of variation of the strength, is ensured for a given characteristic strength, this would not justify reducing the γ_M-values, because this would imply also a lower mean strength. However, see subsection 1.6.1 for abnormal cases. See also subsection 14.1.3 for the cases where the conversion factor η included in γ_c may be reduced.

The γ_M factors applicable to other fundamental variables are given in the relevant clauses.

Numerical values given in Table 1.6.2 include conversion factors η, which, for some applications (see clauses in which these factors should not be taken into account) may be assumed to be equal to 1.1 for γ_c and to 1.0 for γ_s.

If, as envisaged in subsection 1.4.1, γ_c factors are substituted by γ_{Rd} and γ_m factors, the following approximation may usually be accepted:

- γ_m should be taken equal to γ_c divided by a partial factor γ_{Ra} equal to 1.1 taking mainly into account the consequences of an imperfect position of the reinforcement;
- γ_{Rd} should include γ_{Ra}.

Strengths may intervene in S_d via stiffnesses and the spatial distribution throughout the structure. They may generally be favourable as well as unfavourable and are not to be considered as fundamental variables.

These rules shall be amended for accidental situations (see (*a*) of clause 1.6.2.5) and if possible simplifications or refinements defined in (*b*) and (*c*) of clause 1.6.2.5 are applied.

Formula (1.6–2) is the more general. Particular cases are mainly those where

- S_d is an under-proportional function of the actions (or the principal of them); in these cases eq. (1.6–1) may be unsafe; or
- the effects of some actions have a sense opposite to the effects of the other actions and are of the same order of magnitude; in these cases eq. (1.6–1) may be too conservative (this may be the case for the isostatic effects of prestress).

This rule (not splitting γ_M into γ_M and γ_{Rd}) is not applicable in design by testing.

For the definition of individual actions, refer to subsections 1.2.1 and 6.2.1 of Bulletin 191.

Prestress should be added in the symbolic combinations, if relevant.

For the application of the γ factors, see clause 1.6.2.4(c). Besides, for $\gamma_{G\,\sup}$, $\gamma_{G\,\inf}$, $\gamma_{P\,\sup}$ and $\gamma_{P\,\inf}$, refer to clause 1.6.2.4 a1.
For the Ψ factors, refer to clause 1.4.2.2(b).

Whenever strengths intervene in the value of the action-effect S_d the associated γ_M values should be taken equal to 1. This rule is not applicable to buckling verifications (see clause 1.6.3.4(c)), in which strengths are important favourable fundamental variables.

(c) Introduction of the partial coefficients into the calculations
In most cases γ_F factors should be applied globally as follows

$$S_d = S\left\{\gamma_G G + \gamma_P P + \gamma_Q\left(Q_{1k} + \sum_{i>1} \Psi_{oi} Q_{ik}\right)\right\} \qquad (1.6\text{–}1)$$

In particular cases, defined in the relevant clauses of other chapters or to be identified by judgement, for persistent or transient situations, this formula may be substituted by

$$S_d = \gamma_{Sd} S\left\{\gamma_g G + \gamma_P P + \gamma_q\left(Q_{1k} + \sum_{i>1} \Psi_{oi} Q_{ik}\right)\right\} \qquad (1.6\text{–}2)$$

where the partial factors should be taken by referring to a1 above.
These two formulae are partially symbolic and should be applied by following in detail the combination rules given in clause 1.6.2.5.
γ_M factors should generally be applied globally.

1.6.2.5. Combinations of actions

(a) General rules
The combinations of design values to be taken into account for applying the equations (1.6–1) and (1.6–2) above are as follows, in symbolic presentation

- fundamental combinations applicable for persistent and transient situations

$$\gamma_{G\,\sup} G_{\sup} + \gamma_{G\,\inf} G_{\inf} + \gamma_{Q1} Q_{1k} + \sum_{i>1} \gamma_{Qi} \Psi_{oi} Q_{ik} \qquad (1.6\text{–}3)$$

- accidental combinations, applicable for accidental situations

$$G_{\sup} + G_{\inf} + (A_d \text{ or } 0) + \Psi_{11} Q_{1k} + \sum_{i>1} \Psi_{2i} Q_{ik} \qquad (1.6\text{–}4)$$

In these combinations

- $G_{\sup}$ and $G_{\inf}$ refer to the unfavourable and favourable parts of the

- Q_{1k} refers to any variable action, one after the other
- A_d denotes the unique accidental action associated with the accidental situation, if this situation is due to this action. If it is due to another event or to a past action, A_d is substituted by 0.

In most cases some variable actions, which obviously are not the leading ones for a given verification, need not be considered as Q_{1k}.

The actions to be included in any combination are only those that are mutually compatible or are considered as such, as an acceptable approximation. Non-simultaneous actions should be considered in the same combination if their effects are simultaneous.

The cases of incompatibility or negligible compatibility are very numerous. They are given in the codes or standards on actions or identified by judgement (e.g. snow and maximum climatic temperature).

(b) Possible simplifications

Other simplifications may be envisaged and discussed, for example by giving directly design combinations for a given set of common variable actions, such as some imposed loads, wind, snow and temperature.

As an approximation to be recognized by judgement, it is frequently sufficient to limit the total number of variable actions to a maximum of three in any fundamental combination and to two in any accidental combination.

Judgement is necessary because the concept of one action is very blurred. For example the actions of wind, snow, water and imposed loads should be considered as different actions, but the imposed loads on different floors should be considered as one action.

Fundamental combinations that are obviously identified as non-critical may be omitted in the calculations.

In many cases Ψ_{oi} factors may be merged with γ_Q, and S_d may then be calculated, for persistent and transient situations, by

This simplification is mainly intended for common buildings.

$$S_d = S\left(\gamma_G G + \gamma_Q \sum_1^n Q_{ik}\right)$$

where

$\gamma_G = 1$ or 1.35 (take the more unfavourable)
$\gamma_Q = 1.5$ for $n = 1$, or 1.35 for $n \geqslant 2$ (take the more unfavourable).

In accidental combinations Ψ_{11} may often be substituted by Ψ_{21} for most or all variable actions, as a judged approximation or because the occurrence of a greater value during the accidental situation is judged to be very unlikely.

Attention is drawn to the risk that an accident results in consequences on variable actions; for example many persons may gather in some places in order to escape during or immediately after an accident.

(c) Possible refinements

In cases where the most likely consequences of a failure do not seem to be exceptionally severe, the following reductions of γ_F factors in fundamental combinations are possible

This may be the case, for example, if a failure should be limited to a small part of the structure.

This introduces one more combination. Attention is drawn to the necessity, in this case, to verify more completely and carefully than usual the serviceability limit states, which may be less covered than usually by ultimate limit state verifications.

In many cases this does not result in important changes of design.

Clause 1.6.3.2 is more generally valid.

In many cases the verifications may be limited to substructures or isolated elements.

In the most general case (before simplifications) the design principle cannot be expressed by an explicit equation, but only by a set of second order differential equations resulting from moment–curvature relationships throughout the structure with the boundary conditions of the structure.

For reinforced and prestressed concrete structures the first possibility is the more frequent if the structure is rather slender and none of its cross-sections is relatively weak by comparison with most cross-sections. If the structure is not very slender, then second order action effects are relatively small and the second possibility may be postulated a priori.

Within some limits relating to slenderness this can be directly presumed and the second-order action effects can be directly assessed (see section 6.6).

- reduce $\gamma_{G\,\mathrm{sup}}$ to 1.2 or, alternatively, Q_{1k} to $\Psi_{01} Q_{1k}$ or

- reduce to 1.2 the γ_Q value applicable to $\Psi_{oi} Q_{ik} (i > 1)$.

1.6.3. ULS of buckling

1.6.3.1. Definition: field of validity

The verifications treated in this section are stability verifications of slender structures made of columns, walls and beams (or of their elements), in which second order effects are important.

The cases in which torsion effects in columns or in beams cannot be neglected, are not completely covered here.

Local buckling of deep beams and shells is not covered in this Model Code.

1.6.3.2. Design principle

Stability should be verified by demonstrating that

- under the most unfavourable load cases and combinations of actions
- giving the materials and joints their design strengths and the associated deformability
- taking into account geometrical imperfections,

a field of stresses exists throughout the structure, which equilibrates the design action effects, second order action effects included.

Whether the positions and directions of the actions are fixed or are modified by the displacements and deformations of the structure shall be recognized and taken into account in this verification.

According to the case, the verification will be concluded by one of the two following possibilities

- either the design actions are smaller than a proportional loading which would result in instability, before the ultimate limit state of resistance is reached in any cross-section (instability is reached when a small increase of the loading results in very large deformations)
- or the design action effects, second order action effects included, are smaller than the action effects due to a proportional loading which would result in exceeding the limit state of resistance in a cross-section

Simplifications consist of approximating the basic formulation to a greater or lesser degree, thus

(*a*) where action effects are not coplanar with a main plane of the structure, or buckling may occur out of this plane, by verifying successively the stability within two perpendicular planes, and then combining the results according to a conventional rule;

(*b*) for some kinds of structures, by verifying separately some parts of them, having calculated the design action effects imposed at their ends (without considering at this stage the second order effects) and possibly after having simplified these action effects (e.g. by substituting different bending moments by equal moments applied at both ends);

(*c*) for some structural elements, by substituting the real final deformed shape by an a priori chosen shape, restricting thus the limit state equation to the calculation of the magnitude of the deformation at one point;

(*d*) by assessing in a rough way the consequences of the creep due to permanent and long-term actions;

(*e*) for slightly slender columns, by using pragmatically calibrated formulae for assessing directly second order effects (as mentioned in clause 1.6.3.2).

The numerical basis for some simplified models may be obtained by a more simplified analysis.

Such an analysis may for example be elastic-type and based on rough approximate assessments of rigidities resulting in an assessment of 'buckling lengths' (i.e. distances between zero-curvature cross-sections) and 'slenderness ratios'. Such lengths and ratios may also be calculated a priori from pragmatically calibrated rules applicable to simple structural forms. Attention is drawn, however, to the fact that the results of such preliminary analyses and direct assessments often depend on the load case and possibly on the combination of actions.

1.6.3.3. Models for the analysis of the structure and its cross-sections

The general type of both analyses is the non-linear type of analysis defined in clause 5.3.2.1.

The model to be used for calculating the design resistances of cross-sections, is the general model for columns defined in clause 6.3.3.4.

For the structural analysis and hence for studying the possibility of 'instability', models to be used generally are simplified models.

1.6.3.4. Values of partial factors

(a) γ_F factors are given the same values as in clause 1.6.2.4(a).
The first formula (eq. (1.6–1)) given in clause 1.6.2.4(c) should normally be used.

The reason for this is the generally overproportional character of the second order action effects.
The 'linearisation procedure' (eq. (1.6–2)) may, however, be used in exceptional cases, for example for highly redundant and slightly slender structures. In this case γ_f should be given a relatively large value.

(b) γ_s factors are given in the same values as in clause 1.6.2.4(b).
(c) In the calculation of resistances of critical regions, γ_c factors applicable to the compressive strength of concrete are given the same values as in clause 1.6.2.4(c).
In calculation of deformations, for the whole structure and for any structural element, γ_c may be decreased to

(i) 1.2 for fundamental combinations
(ii) 1 for accidental combinations.

Depending on the models used in chapter 6, a γ_E factor on E_c may have also to be considered (see e.g. clause 6.6.2.3).

This reduction may be justified, in spite of the unfavourable character of low strengths, for two reasons:

- the conversion factor of the strength from a standardized specimen to a structural element of any shape, included in γ_c, is not applicable to deformability,
- a low mean strength for a whole structure or element is less likely than for one cross-section.

1.6.3.5. Geometrical imperfections

The corresponding models and magnitudes to be used in the most common cases are defined in section 6.6.

It should be considered whether such values include some real deformations due for example to shrinkage or temperature differences, and whether they may be modified according to tolerances and degree of control.

1.6.4. ULS of fatigue

1.6.4.1. Definition

Fatigue damage occurs through repeated applications or variations of actions (mainly of loads).

Fatigue damage consists of gradual crack propagation in structural parts.
Low cycle (oligocyclic) fatigue, due to less than 10^4 repetitions of actions, is not covered by fatigue limit states defined in this clause and in section 6.7. Although related to service conditions, fatigue limit states are limit states in their own right.

Ultimate limit states of fatigue may be associated with the failure of reinforcing steel, of prestressing steel or of concrete.

In cases where highly alternated tension and relatively high compression occur (e.g. some towers, marine structures or crane girders) fatigue verifications of concrete are necessary. Bond fatigue is also physically possible in some cases, but it is normally covered by the other verifications if the bond properties of the steels are normal.

1.6.4.2. Design principle

Fatigue design shall ensure that in any fatigue endangered cross-section the expected damage D will not exceed a limiting damage $D_{\lim}$.
The verifications of this requirement can be performed according to fou

Maintenance and redesign calculations should take into account the past and expected repetitions of loads.

(a) First method
This is a qualitative verification that no variable action is able to produce fatigue. If the conclusion of this verification is not positive, a verification according to one of the following methods shall be made.

Static actions not repeated more than 10^4 times or for which $\Psi_1 = 0$ are considered unable to produce fatigue. Examples of actions able to cause fatigue are loads due to vehicles, cranes, moving machinery, wind (gusts, turbulence, vortices, etc.) and wave action.

(b) Second method
This is a verification that

- for steel the maximum design stress range $\gamma_{Sd}\Delta\sigma_s(G,\ P,\ \Psi_1 Q_k)$
- for concrete in compression the maximum compressive stress $\gamma_{Sd}\sigma_{c,\max}(G,\ P,\ \Psi_1 Q_k)$
- for plain concrete in tension the maximum design tensile stress $\gamma_{Sd}\sigma_{ct,max}(G,\ P,\ \Psi_1 Q_k)$

do not exceed the values given in subsection 6.7.3.

This is an indirect verification that the loss of strength will not be significant. The representative value of P should be chosen such that the unfavourable situation is covered.

In assessing the stress range, stress variations in opposite senses (due for example to successive arrangements of a moveable load) shall be, if relevant, taken into account.

Other design properties associated with the tensile stress of concrete (e.g. a formal shear stress) may also have to be considered.

(c) Third method
This verification refers to a representation of the variable load dominant for fatigue by a single magnitude Q associated with a number of repetitions n during the required lifetime.

This single value of Q may either correspond to identical magnitudes in all applications, or have fatigue effects equivalent to the effects of the action with its actual magnitudes.

The stresses (or stress range) due to the application of Q (possibly due to applications in two senses or due to successive load arrangements) are multiplied by a factor γ_{Sd} given in 1.6.4.4. These design values shall be smaller than the resistances to fatigue for n cycles, as defined in sub-section 6.7.4, divided by a specific γ_M-factor ($\gamma_{s,fat}$ or $\gamma_{c,fat}$ depending on the material) also given in 1.6.4.4.

The single value of Q may result into stress ranges in various cases, e.g.:
- the action is fixed but intermittent
- the action is fixed and alternate
- the action is moveable.

This value—as a fatigue equivalent one—should be taken as far as possible from structural codes or codes on actions on structures. In many cases the frequent value $\Psi_1 Q_k$ may be used as approximately equivalent or safe-sided.

The limit state equations, depending on the material, are defined in subsection 6.7.4.

For the assessment of the stresses and stress ranges, see (b); the frequent value of another variable action, e.g. temperature, is taken into account if relevant (i.e. if it increases the stress range in the case of non-linear behaviour).

For steel, for example, the limit state equation may be written

$$\gamma_{Sd} \max \Delta\sigma_{Ss}(G,\ P,\ Q,\ \Psi_1 T_k) \leqslant \Delta\sigma_{Rsk}(n)/\gamma_{s,fat}$$

The condition may also be presented as $n < N$ when N is the resisting number of applications of a stress range equal to $\gamma_{s,fat}\gamma_{Sd}\Delta\sigma_s$.

(d) Fourth method
This is a verification based on an assessment of the fatigue damage resulting from various magnitudes of loads. The load history during the required life should usually be represented by a spectrum in a discretized form. The accumulation of fatigue damage is calculated on the basis of the Palmgren–Miner summation.

1.6.4.3. Models for the analysis of the structure and its cross-sections

Linear elastic models generally may be used, and reinforced concrete in tension is considered to be cracked.

1.6.4.4. Values of partial factors

The γ-factors have the following numerical values

$$\gamma_{Sd} = 1.1$$
$$\gamma_{c,fat} = \gamma_c = 1.5$$
$$\gamma_{s,fat} = \gamma_s = 1.15$$

If the stress analysis is sufficiently accurate or conservative, and this fact is verified by in-situ observations, it may be possible to take $\gamma_{Sd} = 1.0$.

This includes, e.g. for bent bars, some conversion factors not included in the γ-factors.

The application of the partial safety factors is shown in section 6.7.

1.6.4.5. Effects of combined actions

In cases with superimposed loads due to different actions, e.g. winds, waves, vehicles, etc., it is necessary, if the verification is performed according to 1.6.4.2(d), to treat them according to whether they are correlated in time or not. If they are correlated, the corresponding stresses should be added; if not, the damages can be added separately.

Permanent actions including prestress should also be taken into account using their representative values.

1.6.5. ULS of static equilibrium and analogous limit states

1.6.5.1. Definition

These limit states are all limit states beyond which the structure, or a part of it, is overturned, slides or is lifted from its support.

With regard to these limit states, the structure, or the part of it under consideration, is considered to behave isostatically.

1.6.5.2. Design principle

In the simplest cases the condition may be written

$$S_{1d} \leqslant S_{2d}$$

where S_1 and S_2 are effects of destabilizing and stabilizing actions, respectively.

For frictional limit states it may be written

$$S_{1d} \leqslant \mu_d S_{2d}$$

where

μ_d is the design value of the coefficient of friction, and
S_1 and S_2 represent tangential and normal forces, respectively.

In other cases other fundamental variables (e.g. some geometrical variables) should be introduced in the limit state equations.

In some cases the overall stability cannot be maintained without a particular element, such as an anchoring device, of the structure. Then the condition should be written

$$S_{1d} \leqslant S_{2d} + R_d$$

where R_d is relatively small in comparison with S_{1d} and S_{2d}.

In these equations the deformations due to actions and possibly to foundation settlements (second order effects) should be taken into account.

Permanent actions, selfweight of the structure included, should be split into two parts: those not favourable to the stability (destabilizing actions) and those favourable to the stability (stabilizing actions). These two parts are used for the calculation of S_1 and S_2, respectively.

Variable and accidental actions generally are considered only when they are destabilizing (i.e. in S_1).

For variable actions specific models should sometimes be used (e.g. free vertical component of a wind pressure).

1.6.5.3. Values of the partial factors and combinations of actions

Design values of variable and accidental actions and combinations of their values are the same as defined in clauses 1.6.2.4 and 1.6.2.5. γ_Q-factors are introduced globally into the calculations.

Reference is made, if relevant, to the second paragraph of clause 1.6.2.4 a1 following Table 1.6.1 (i.e. taking into account the ξ factor).

For permanent actions $\gamma_{G\,\mathrm{sup}}$ and $\gamma_{G\,\mathrm{inf}}$ are respectively applied to the permanent actions included in S_1 and S_2, and should depend on

- the variability of these actions
- the possible correlation between them.

The γ_G-values given in this clause are associated with the representative values of permanent actions defined in clause 1.4.2.2.

In most cases, for cast-in-situ structures $\gamma_{G\,\mathrm{inf}}$ may be taken equal to 0.95 and $\gamma_{G\,\mathrm{sup}}$ to 1.05 if the selfweight of the structure is the major part of the permanent actions.

For prefabricated elements, values closer to 1 may be accepted. The same happens for accidental situations.

γ_G-values may also be taken closer to 1 for verifications during transient situations under control (e.g. the lower support reaction is measured before application of the whole destabilizing load). Such controls are always recommended when values closer to 1 than values given in the text are adopted.

If, as envisaged in clause 1.6.5.2, the stability involves a resistance R_d, the values 0.9 and 1.1 of γ_G-factors are substituted respectively by 1.1 and 1.35, and the values 0.95 and 1.05 by 1.2 and 1.35.

They often represent the main cause of variability in such limit states. This is especially the case for friction coefficients and geometrical variables.

Design resistances R_d, if they intervene, as envisaged in clause 1.6.5.2, are given the same design values as for ultimate limit states of resistance.

For concrete structures the limit states of cracking defined in section 7.4 may have a paramount importance. Deformation limit states mainly depend on the type of building or civil engineering works and on its equipment and use.

As mentioned in subsections 7.4.4, 7.4.5 and 7.3.4 some of these rules may in some cases be substituted by stress limitations, detailing rules or other indirect verifications.

The α-factor (e.g. 0.6 for excessive compression) describes the limit state and is not a reliability factor.

In such equations f generally is not to be considered as a fundamental variable.

If the correlation between stabilizing and destabilizing actions is not relatively high, $\gamma_{G\,\mathrm{inf}}$ should be taken equal to 0.9 and $\gamma_{G\,\mathrm{sup}}$ equal to 1.1 for persistent and transient situations. For accidental situations they should be taken equal to 1.

Design values of the other design variables should be chosen carefully where they are judged to be treated as fundamental variables.

1.6.6. Serviceability limit states

1.6.6.1. Definition and classification

These limit states have been defined and classified in subsection 1.2.3. They are treated in detail in chapter 7.

1.6.6.2. Design principle

(a) Limit state of cracking and excessive compression

It should be verified that in any cross-section

$\sigma(F_d) < \alpha f_d$ for crack formation and excessive creep effects

$w(F_d, f) < w_{\mathrm{lim}}$ for maximum crack width

$\sigma(F_d) \leqslant 0$ for crack re-opening

where

σ is a defined stress
f_d is a tensile, shear or compressive design strength
w is a defined crack width.

(b) Limit state of deformations
It should be verified that

$$a(F_d, f_d) \leqslant C_d$$

where a is a defined deformation (generally a deflection).

This rule may in some cases be substituted by a maximum slenderness ratio.

If not fixed by the Code, C_d should be fixed by the contract or chosen by the designer, possibly depending on non-structural parts.

(c) Limitation of vibrations
In the most common cases the limitation is ensured by indirect measures, such as limiting the deformations or the periods of vibration of the structure in order to avoid the risk of resonance. In the other cases a dynamic analysis is necessary.

See section 7.6.

1.6.6.3. Models for the analysis of the structure and its cross-sections

Elastic analysis is normally used. Non-linear analysis may be used.
The possibility of cracks should be considered.
Models for the analysis are defined in chapters 5 and 7.

Second order action effectss should be considered in very particular cases.

1.6.6.4. Values of partial factors

(a) γ_F-factors are taken equal to 1.
(b) γ_M-factors are taken equal to 1.

Pragmatic values smaller than 1 may be envisaged for indirect actions.

1.6.6.5. Combination of actions

(a) General rules
The combinations which should be considered depend on the particular limit state under consideration and are identified in the corresponding chapters.

They are defined as follows, in a symbolic presentation

rare: $G + P + Q_{1k} + \sum_{i>1} (\Psi_{oi} Q_{ik})$

frequent: $G + P + \Psi_{11} Q_{1k} + \sum_{i>1} (\Psi_{2i} Q_{ik})$

quasi-permanent: $G + P + \sum_{i \geqslant 1} (\Psi_{2i} Q_{ik})$

where G is taken according to clause 1.4.2.2 and Q_1 refers to any variable action, successively.

The appropriate representative value of P (i.e. P_m), where P_k should not be used, is specified in chapter 7.

(b) Possible simplification

The first two paragraphs of clause 1.6.2.5(b) may be applied to combinations for serviceability limit states.

In common cases for reinforced concrete structures, the rare combinations may be simplified by avoiding reference to various Ψ_{oi} factors. They are substituted, in a symbolic presentation, by

$$G + Q_{1k}$$

or

$$G + 0.9 \sum_{1}^{n} Q_{ik} \text{ (take the more unfavourable)}$$

in which Q_{1k} is the most unfavourable variable action.

This simplification is analogous to the simplification of the fundamental combinations defined in 1.6.2.5(b). If the most unfavourable variable action is easily identified, the number of these combinations is reduced to 2 for any number of variable actions.

Substituting frequent combinations by these combinations is another possible simplification which however may be excessively conservative, and not useful in many cases where the dominating frequent combination, among the n possible combinations, can be easily identified.

2. MATERIAL PROPERTIES

2.1. CONCRETE CLASSIFICATION AND CONSTITUTIVE RELATIONS

2.1.1. Definitions and classification

2.1.1.1. Range of applicability

The subsequent clauses apply to concrete with normal weight aggregates so composed and compacted as to retain no appreciable amount of entrapped air other than intentionally entrained air.

Though the relations given in the subsequent sections in principle also apply for heavyweight concrete, special consideration may be necessary for such concretes.

For technological aspects including the production of lightweight aggregate concrete refer to Appendix d. The constitutive relations given in these sections are applicable for the entire range of concrete grades dealt with in this Model Code. However, since the available information on the behaviour of concrete with a characteristic strength higher than 50 MPa is rather limited, the constitutive relations should be used with caution for $f_{ck} > 50$ MPa.

Note that throughout this section the following sign conventions are maintained which may differ from those used in other parts of this Model Code.

- Material properties are positive or to be used in absolute terms, e.g. compressive strength, $f_{cm} = |f_{cm}|$.
- Tensile stresses and tensile strains (elongations) are positive.
- Compressive stresses and compressive strains (contractions) are negative.
- Where multiaxial stress states are considered, $\sigma_1 > \sigma_2 > \sigma_3$.

2.1.1.2. Classification by strength — concrete grades

In this Model Code concrete is classified on the basis of its compressive strength. Design is based on a grade of concrete which corresponds to a specific value of its characteristic compressive strength f_{ck} as defined in clause 2.1.3.2.

Concrete grades for normal weight concrete can be selected from the following series:

<u>C12</u> C16 <u>C20</u> C25 <u>C30</u> C35 <u>C40</u> C45 <u>C50</u> C55 <u>C60</u> C65 <u>C70</u> C75 <u>C80</u>

where the numbers denote the specified characteristic compressive strength f_{ck} in MPa. For production and quality control reasons concrete should be specified in steps of 10 MPa, and the values underline are recommended.

With regard to classification on the basis of density or durability refer to d.5.3 in Appendix d.

For reinforced concrete only grades C16 and above should be used. For prestressed concrete only grades C25 and above should be used.

2.1.2. Density

For normal weight concrete the following values of density may be used in design calculations

Where a higher accuracy is required concrete density may be determined experimentally e.g. according to ISO 6275.

$\rho = 2400\,kg/m^3$ for plain concrete
$\rho = 2500\,kg/m^3$ for reinforced and prestressed concrete.

2.1.3. Strength

2.1.3.1. Range of applicability

The information given in this section is valid for monotonically increasing compressive stresses or strains at a rate of $|\dot{\sigma}_c| \sim 1.0\,MPa/s$ or $|\dot{\varepsilon}_c| \sim 30 \times 10^{-6}\,s^{-1}$, respectively. For tensile stresses or strains it is valid for $\dot{\sigma}_{ct} \sim 0.1\,MPa/s$ or $\dot{\varepsilon}_{ct} \sim 3.3 \times 10^{-6}\,s^{-1}$, respectively.

2.1.3.2. Compressive strength

This Code is based on the uniaxial compressive strength f_c of cylinders, 150 mm in diameter and 300 mm in height stored in water at $20 \pm 2°C$, and tested at the age of 28 days in accordance with ISO 1920, ISO 2736/2 and ISO 4012.

For special requirements or in national codes test specimens other than cylinders 150/300 mm and stored in other environments may be used to specify concrete compressive strength. In such cases conversion factors should be determined by direct tests or as given in national codes for a given category of testing equipment.

The characteristic compressive strength f_{ck} (MPa) is defined as that strength below which 5% of all possible strength measurements for the specified concrete may be expected to fall.

For some verifications in design or for an estimate of other concrete properties it is necessary to refer to a mean value of compressive strength f_{cm} associated with a specific characteristic compressive strength f_{ck}. In this case f_{cm} may be estimated from eq. (2.1–1):

$$f_{cm} = f_{ck} + \Delta f \tag{2.1–1}$$

where $\Delta f = 8\,MPa$.

In practice, the concrete is regarded to comply with the grade specified for the design if the test results comply with the acceptance criteria given in Chapter 12.

Where concrete cubes 150/150/150 mm are used, the characteristic strength values given in Table 2.1.1 shall be obtained for the various concrete grades.

Table 2.1.1. Characteristic strength values (MPa)

Concrete grade	C12	C20	C30	C40	C50	C60	C70	C80
f_{ck}-cylinder	12	20	30	40	50	60	70	80
f_{ck}-cube	15	25	37	50	60	70	80	90

2.1.3.3. Tensile strength and fracture properties

2.1.3.3.1. Tensile strength

In this Code, unless stated otherwise, the term 'tensile strength' refers to the axial tensile strength f_{ct} determined in accordance with RILEM CPC 7

The tensile strength of concrete is more variable than its compressive strength. It is influenced by the shape and the surface texture of the aggregates more than the compressive strength and may be reduced substantially by environmental effects. Therefore, the tensile strength of concrete should be taken into account in design with caution.

In absence of more accurate data for a particular concrete the lower and upper bound values of the characteristic tensile strength $f_{ctk,\max}$ and $f_{ctk,\min}$ may be estimated from the characteristic compressive strength using eqs (2.1–2) and (2.1–3)

$$f_{ctk,\min} = f_{ctko,\min}\left(\frac{f_{ck}}{f_{cko}}\right)^{2/3} \qquad (2.1\text{–}2)$$

$$f_{ctk,\max} = f_{ctko,\max}\left(\frac{f_{ck}}{f_{cko}}\right)^{2/3} \qquad (2.1\text{–}3)$$

where

$$f_{cko} = 10\,\mathrm{MPa}$$

$$f_{ctko,\min} = 0.95\,\mathrm{MPa}$$

$$f_{ctko,\max} = 1.85\,\mathrm{MPa}.$$

For some verifications in design or for an estimate of other concrete properties it is necessary to refer to a mean value of tensile strength f_{ctm} associated with a specified characteristic compressive strength f_{ck}. In this case f_{ctm} may be estimated from eq. (2.1–4)

$$f_{ctm} = f_{ctko,m}\left(\frac{f_{ck}}{f_{cko}}\right)^{2/3} \qquad (2.1\text{–}4)$$

where $f_{ctko,m} = 1.40\,\mathrm{MPa}$.

The corresponding values for the characteristic tensile strength of different concrete grades are given in Table 2.1.2.

Table 2.1.2. Tensile strength for various concrete grades (MPa)

Concrete grade	C12	C20	C30	C40	C50	C60	C70	C80
f_{ck}	12	20	30	40	50	60	70	80
f_{ctm}	1.6	2.2	2.9	3.5	4.1	4.6	5.1	5.6
$f_{ctk,\min}$	1.1	1.5	2.0	2.4	2.8	3.1	3.5	3.8
$f_{ctk,\max}$	2.1	2.9	3.8	4.7	5.4	6.1	6.8	7.4

Though both f_{ctm} and $f_{ct,sp}$ depend on the size of the specimen, this size effect is in the range of the effect of specimen size on compressive strength. The effect of the depth of a beam on flexural tensile strength is more pronounced. It may be calculated on the basis of fracture characteristics, given in clauses 2.1.3.3.2 and 2.1.4.4.2 or estimated from eq. (2.1–6).

If the tensile strength is measured as splitting tensile strength $f_{ct,sp}$ or as flexural tensile strength $f_{ct,fl}$ conversion factors should be determined by means of direct tests.

If such conversion factors are not available the mean axial tensile strength f_{ctm} may be estimated from the mean splitting tensile strength $f_{ct,sp}$ according to eq. (2.1–5)

$$f_{ctm} = 0.9 f_{ct,sp} \qquad (2.1\text{–}5)$$

where

$f_{ct,sp}$ is the mean value of splitting tensile strength determined according to ISO 4108,

f_{ctm} is the mean value of axial tensile strength; it may be estimated from the mean flexural tensile strength according to eq. (2.1–6)

Eq. (2.1–6) is an approximation neglecting the effect of maximum aggregate size. It is valid for $h_b > 50\,\text{mm}$.

$$f_{ctm} = f_{ct,fl} \frac{1.5\,(h_b/h_o)^{0.7}}{1 + 1.5\,(h_b/h_o)^{0.7}} \qquad (2.1\text{–}6)$$

where

$f_{ct,fl}$ is the mean value of flexural tensile strength determined according to ISO 4013

h_b is the depth of beam (mm)

h_o = 100 mm.

According to the RILEM Draft Recommendation TC50-FMC the fracture energy G_F is determined on notched specimens loaded in flexure. G_F corresponds to the area under the load-deflection relationship divided by the net cross-section of the specimen above the notch (see also clause 2.1.4.4.2).

With regard to the formulation of fracture properties refer to: H.K. Hilsdorf, W. Brameshuber, 'Code-Type Formulation of Fracture Mechanics Concepts for Concrete', International Journal of Fracture, Vol. 51, pp. 61–72, 1991.

2.1.3.3.2. Fracture energy

The fracture energy of concrete G_F is the energy required to propagate a tensile crack of unit area.

In the absence of experimental data G_F may be estimated from eq. (2.1–7):

$$G_F = G_{Fo}(f_{cm}/f_{cmo})^{0.7} \qquad (2.1\text{–}7)$$

where

f_{cmo} = 10 MPa.

G_{Fo} is the base value of fracture energy. It depends on the maximum

Fracture energy G_F does, to some extent, depend on the size of the structural member as well as on other concrete properties not taken into account in eq. (2.1–7), resulting in deviations of G_F from the values according to eq. (2.1–7) of up to $\pm 30\%$.

Table 2.1.3. Base values of fracture energy G_{Fo} (Nmm/mm²)

d_{max} (mm)	G_{Fo} (Nmm/mm²)
8	0.025
16	0.030
32	0.058

The corresponding values for G_F for different concrete grades may also be taken from Table 2.1.4.

Note that in Table 2.1.4 G_F is given in (Nm/m²) whereas from eq. (2.1–7) and Table 2.1.3 values of G_F in (Nmm/mm²) are obtained.

Table 2.1.4. Fracture energy G_F (Nm/m²)

Max. aggregate size d_{max} (mm)	G_F (Nm/m²)							
	C12	C20	C30	C40	C50	C60	C70	C80
8	40	50	65	70	85	95	105	115
16	50	60	75	90	105	115	125	135
32	60	80	95	115	130	145	160	175

2.1.3.4. Strength under multiaxial states of stress

The strength of concrete under multiaxial states of stress may be estimated from the failure criterion given by eq. (2.1–8):

$$\alpha \frac{J_2}{f_{cm}^2} + \lambda \frac{\sqrt{J_2}}{f_{cm}} + \beta \frac{I_1}{f_{cm}} - 1 = 0 \qquad (2.1\text{–}8)$$

where

$$\lambda = c_1 \cos[1/3 \arccos(c_2 \cos 3\theta)] \qquad \text{for } \cos 3\theta \geqslant 0 \qquad (2.1\text{–}9a)$$

$$\lambda = c_1 \cos[\pi/3 - 1/3 \arccos(-c_2 \cos 3\theta)] \qquad \text{for } \cos 3\theta < 0 \qquad (2.1\text{–}9b)$$

$$\cos 3\theta = \frac{3\sqrt{3}}{2} \frac{J_3}{J_2^{3/2}} \qquad (2.1\text{–}10)$$

This failure criterion is one among several acceptable formulations. It has been chosen since it is not too difficult to use and agrees well with test data. For further details and the range of applicability of eq. (2.1–8) refer to 'Concrete under multiaxial states of stress—constitutive equations for practical design', CEB Bulletin 156, Lausanne, 1983 and to Ottosen, N., 'A Failure Criterion for Concrete', Journal Engineering Mechanics Division, ASCE, Vol. 103, EM4, August 1977.

The stress tensor (I_1) and the stress deviators (J_2 and J_3) used in eqs (2.1–8) to (2.1–10) may be calculated as follows

$$I_1 = \sigma_1 + \sigma_2 + \sigma_3$$

$$J_2 = \tfrac{1}{6}[(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2]$$

$$J_3 = (\sigma_1 - \sigma_m)(\sigma_2 - \sigma_m)(\sigma_3 - \sigma_m)$$

$$\sigma_m = (\sigma_1 + \sigma_2 + \sigma_3)/3$$

where σ_1, σ_2 and σ_3 are the principal stresses.

The strength ratio k may be estimated using eqs (2.1–1) and (2.1–4).

According to this failure criterion a given state of stress does not lead to failure if the left side of eq. (2.1–8) is negative.

The failure criterion is derived on the assumption that the biaxial compressive strength $f_{2cm} = 1.2 f_{cm}$.

For further information refer to: Kupfer, H.B., Gerstle, K.H., 'Behaviour of Concrete under Biaxial Stresses', Journal Engineering Mechanics Division, ASCE, Vol. 99, EM4, August 1973.

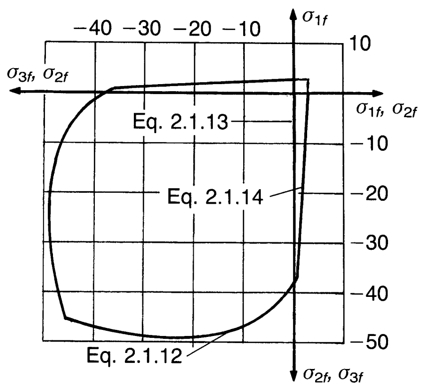


Fig. 2.1.1. Biaxial strength of concrete C30 according to eqs (2.1–12) to (2.1–14)

The parameters J_2, J_3 and I_1 in eqs (2.1–8) to (2.1–10) represent the invariants of the stress deviator and stress tensor, respectively, characterizing the state of stress considered.

The coefficients α, β, c_1 and c_2 are material parameters which depend on the strength ratio $k = f_{ctm}/f_{cm}$

$$\left.\begin{aligned} \alpha &= \frac{1}{9k^{1.4}} \qquad & \beta &= \frac{1}{3.7k^{1.1}} \\ c_1 &= \frac{1}{0.7k^{0.9}} \qquad & c_2 &= 1 - 6.8(k - 0.07)^2 \end{aligned}\right\} \tag{2.1–11}$$

The strength of concrete under biaxial states of stress may be estimated from the simplified criteria, given in eqs (2.1–12) to (2.1–14). They are in acceptable agreement with the more realistic predictions from eqs (2.1–8) to (2.1–11).

Biaxial compression and tension-compression for $\sigma_{3f} < -0.96 f_{cm}$

$$\sigma_{3f} = -\frac{1 + 3.80\alpha}{(1 + \alpha)^2} f_{cm} \tag{2.1–12}$$

where $\alpha = \sigma_{2f}/\sigma_{3f}$.

Biaxial tension

$$\sigma_{1f} = f_{ctm} = \text{const.} \tag{2.1–13}$$

Biaxial tension-compression for $\sigma_{3f} > -0.96 f_{cm}$

$$\sigma_{1f} = \left(1 + 0.8\frac{\sigma_{3f}}{f_{cm}}\right) f_{ctm} \tag{2.1–14}$$

where

σ_{1f} is the largest principal stress at failure

σ_{2f} is the intermediate principal stress at failure

2.1.4. Stress and strain

2.1.4.1. Range of application

The information given in this section is valid for monotonically increasing compressive stresses or strains at a rate of $|\dot{\sigma}_c| \sim 30\,\mathrm{MPa/s}$ or $|\dot{\varepsilon}_c| \sim 30 \times 10^{-6}\,\mathrm{s}^{-1}$, respectively. For tensile stresses or strains it is valid for $\dot{\sigma}_{ct} \sim 0.03\,\mathrm{MPa/s}$ or $\dot{\varepsilon}_{ct} \sim 3 \times 10^{-6}\,\mathrm{s}^{-1}$, respectively.

2.1.4.2. Modulus of elasticity

Values of the modulus of elasticity for normal weight concrete can be estimated from the specified characteristic strength using eq. (2.1–15)

$$E_{ci} = E_{co}[(f_{ck} + \Delta f)/f_{cmo}]^{1/3} \qquad (2.1\text{–}15)$$

where

E_{ci} is the modulus of elasticity (MPa) at a concrete age of 28 days
f_{ck} is the characteristic strength (MPa) according to clause 2.1.3.2
$\Delta f = 8\,\mathrm{MPa}$
$f_{cmo} = 10\,\mathrm{MPa}$
$E_{co} = 2.15 \times 10^4\,\mathrm{MPa}$.

Where the actual compressive strength of concrete at an age of 28 days f_{cm} is known, E_{ci} may be estimated from eq. (2.1–16)

$$E_{ci} = E_{co}[f_{cm}/f_{cmo}]^{1/3} \qquad (2.1\text{–}16)$$

Where only an elastic analysis of a concrete structure is carried out, a reduced modulus of elasticity E_c according to eq. (2.1–17) should be used in order to account for the initial plastic strain

$$E_c = 0.85E_{ci} \qquad (2.1\text{–}17)$$

Values of the tangent moduli E_{ci} and the reduced moduli E_c for different concrete grades are given in Table 2.1.6.

Table 2.1.6. Tangent moduli and reduced moduli of elasticity

Concrete grade	C12	C20	C30	C40	C50	C60	C70	C80
E_{ci} (10^3 MPa)	27	30	34	36	39	41	43	44
E_c (10^3 MPa)	23	26	29	31	33	35	36	38

The modulus of elasticity as obtained from eqs (2.1–15) and (2.1–16), respectively, is defined as the tangent modulus of elasticity at the origin of the stress–strain diagram. It is approximately equal to the slope of the secant of the unloading branch for rapid unloading and does not include initial plastic deformations. It has to be used for the description of stress–strain diagrams for uniaxial compression, uniaxial tension and multiaxial stress–states according to eqs (2.1–18) to (2.1–22), (2.1–23) and (2.1–29), respectively, as well as for an estimate of creep according to eqs (2.1–61) and (2.1–62). The reduced modulus of elasticity E_c according to eq. (2.1–17) includes some irreversible strains.

Even for a given strength the modulus of elasticity depends on the type of aggregates. Eq. (2.1–15) is valid for concretes made of quartzitic aggregates. For concrete made of basalt, dense limestone, limestone or sandstone the modulus of elasticity according to eq. (2.1–15) may be calculated by multiplying E_{ci} with the coefficients α_E from Table 2.1.5.

Table 2.1.5. Effect of type of aggregate on modulus of elasticity

Aggregate type	α_E
Basalt, dense limestone aggregates	1.2
Quartzitic aggregates	1.0
Limestone aggregates	0.9
Sandstone aggregates	0.7

To take full account of differences in aggregate stiffness or modulus, direct measurements of E_{ci} are necessary.

2.1.4.3. Poisson's ratio

For a range of stresses $-0.5 f_{ck} < \sigma_c < f_{ctk}$ Poisson's ratio of concrete, ν_c, is between 0.1 and 0.2.

2.1.4.4. Stress–strain relations for short-term loading

2.1.4.4.1. Compression

The stress–strain diagrams are generally of the form shown schematically in Fig. 2.1.2.

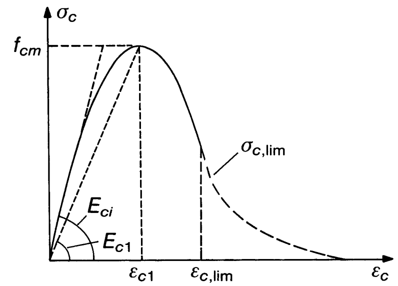


Fig. 2.1.2. Stress–strain diagram for uniaxial compression

The descending portion of the stress–strain relations should be considered as the envelope to all possible stress–strain relations of a concrete which tends to soften as a consequence of concrete micro-cracking.

Several relations exist to describe stress–strain relationships for concrete in compression. Among the suitable ones is the relation given by eq. (2.1–18). It can also be used as a basis to calculate stress–strain diagrams under multiaxial states of stress (clause 2.1.4.4.3).

Similar to tensile failure also compression failure of concrete is often a discrete phenomenon, i.e. there is a fracture region of limited width, in which compression strains are concentrated.

For practical reasons and due to lack of sufficient experimental data these strain concentrations generally are smeared as has been done in eqs (2.1–18) to (2.1–21). As a consequence, the descending branch of the stress–strain relation in compression is influenced by the length of the member subjected to compression, as can be seen from Fig. 2.1.3. Eqs (2.1–18) to (2.1–21) are reasonably accurate for a length of the member subjected to compression of approximately 200 mm.

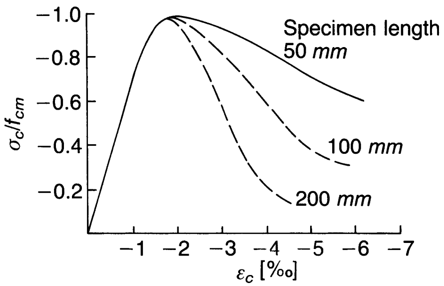


Fig. 2.1.3. Influence of specimen length on the uniaxial stress–strain curve for a constant cross-section 100 × 100 mm^2 *(van Mier, J.G.M., Multiaxial strain-softening of concrete, Materials and Structures, No. 111, May/June*

The $\sigma_c - \varepsilon_c$-relationship may be approximated by the following function:

$$\sigma_c = -\frac{\dfrac{E_{ci}}{E_{c1}}\dfrac{\varepsilon_c}{\varepsilon_{c1}} - \left(\dfrac{\varepsilon_c}{\varepsilon_{c1}}\right)^2}{1 + \left(\dfrac{E_{ci}}{E_{c1}} - 2\right)\dfrac{\varepsilon_c}{\varepsilon_{c1}}} f_{cm} \quad \text{for } |\varepsilon_c| < |\varepsilon_{c,\lim}| \qquad (2.1\text{–}18)$$

where

E_{ci} is the tangent modulus according to eq. (2.1–16)
σ_c is the compression stress (MPa)
ε_c is the compression strain
$\varepsilon_{c1} = -0.0022$
$E_{c1} = f_{cm}/0.0022$ = secant modulus from the origin to the peak

The size dependence of the stress–strain relation for concrete in compression is responsible for various size effects observed in the behaviour of reinforced concrete elements. Also refer to Hillerborg, A., 'The compression stress-strain curve for design of reinforced concrete beams', Fracture Mechanics: Application to Concrete, American Concrete Institute, ACI-SP-118, Detroit, Michigan, 1989.

For the descending part of the stress–strain diagram eq. (2.1–18) is valid only for values of $|\sigma_c|/f_{cm} \geqslant 0.5$.

The strain $\varepsilon_{c,\lim}$ at $\sigma_{c,\lim} = -0.5 f_{cm}$ may be calculated from eq. (2.1–19)

$$\frac{\varepsilon_{c,\lim}}{\varepsilon_{c1}} = \frac{1}{2}\left(\frac{1}{2}\frac{E_{ci}}{E_{c1}} + 1\right) + \left[\frac{1}{4}\left(\frac{1}{2}\frac{E_{ci}}{E_{c1}} + 1\right)^2 - \frac{1}{2}\right]^{1/2} \qquad (2.1\text{–}19)$$

Values for E_{ci}, E_{c1} and $\varepsilon_{c,\lim}$ for various concrete grades are given in Table 2.1.7.

Table 2.1.7. E_{ci}, E_{c1} and $\varepsilon_{c,\lim}$ for various concrete grades

Concrete grade	C12	C20	C30	C40	C50	C60	C70	C80
E_{ci} (10^3 MPa)	27	30.5	33.5	36.5	38.5	41	42.5	44.5
E_{c1} (10^3 MPa)	9	12.5	17.5	22	26.5	31	35.5	40
$\varepsilon_{c,\lim}$ (10^{-3})	−5.0	−4.2	−3.7	−3.3	−3.0	−2.8	−2.6	−2.4

The stress–strain relation according to eq. (2.1–18) is valid for monotonically increasing strains at a rate of approximately $-30 \times 10^{-6}\,\mathrm{s}^{-1}$. For substantially slower rates such as may occur during construction, stress–strain relations up to a stress $|\sigma_c| < 0.6 f_{ck}$ may be estimated from an incremental stress increases taking into account creep according to clause 2.1.6.4.3.

The strain $\varepsilon_{c,\lim}$ has no significance other than limiting the applicability of eq. (2.1–18).

For strains $|\varepsilon_c| > |\varepsilon_{c,\lim}|$ the descending branch of the $\sigma_c - \varepsilon_c$ diagram may be described using eqs (2.1–20) and (2.1–21):

$$\sigma_c = -\left[\left(\frac{1}{\varepsilon_{c,\lim}/\varepsilon_{c1}}\xi - \frac{2}{(\varepsilon_{c,\lim}/\varepsilon_{c1})^2}\right)\left(\frac{\varepsilon_c}{\varepsilon_{c1}}\right)^2 + \left(\frac{4}{\varepsilon_{c,\lim}/\varepsilon_{c1}} - \xi\right)\frac{\varepsilon_c}{\varepsilon_{c1}}\right]^{-1} f_{cm} \qquad (2.1\text{–}20)$$

with

$$\xi = \frac{4\left[\left(\frac{\varepsilon_{c,\lim}}{\varepsilon_{c1}}\right)^2\left(\frac{E_{ci}}{E_{c1}} - 2\right) + 2\frac{\varepsilon_{c,\lim}}{\varepsilon_{c1}} - \frac{E_{ci}}{E_{c1}}\right]}{\left[\frac{\varepsilon_{c,\lim}}{\varepsilon_{c1}}\left(\frac{E_{ci}}{E_{c1}} - 2\right) + 1\right]^2} \qquad (2.1\text{–}21)$$

As a simplifying alternative the descending branch of the $\sigma_c - \varepsilon_c$ diagram may also be approximated by a straight line according to eq. (2.1–38) and as shown in Fig. 2.1.8.

For cross-section design an idealized parabola-rectangle $\sigma_c - \varepsilon_c$-diagram may be used (see clause 6.2.2.2).

The stress–strain relation for unloading of the uncracked concrete may be described by eq. (2.1–22)

$$\Delta\sigma_c = E_{ci}\Delta\varepsilon_c \qquad (2.1\text{–}22)$$

where

$\Delta\sigma_c$ is the stress reduction
$\Delta\varepsilon_c$ is the strain reduction.

Figure 2.1.4 shows examples of stress–strain diagrams for concrete in uniaxial compression as derived from eqs (2.1–18) through (2.1–21).

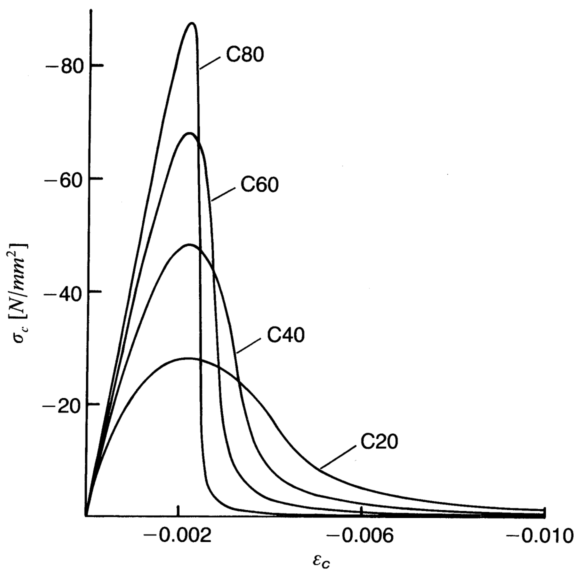


Fig. 2.1.4. Stress–strain diagrams for concrete in compression according to eqs 2.1–18 through 2.1–21

Tensile failure of concrete is always a discrete phenomenon. Therefore, to describe the tensile behaviour a stress–strain diagram should be used for the uncracked concrete, and a stress-crack opening diagram as shown in Fig. 2.1.5 should be used for the cracked section.

2.1.4.4.2. Tension

For uncracked concrete subjected to tension a bilinear stress–strain relation as given in eqs (2.1–23) and (2.1–24) may be used.

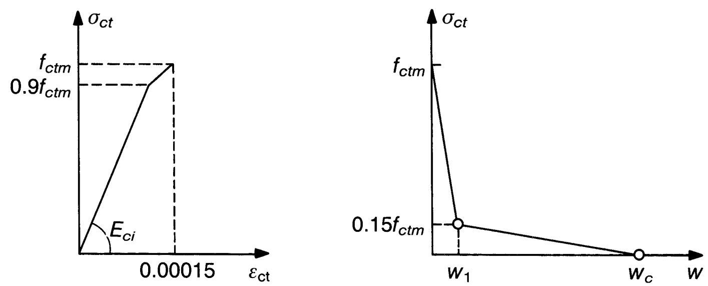


Fig. 2.1.5. Stress–strain and stress-crack opening diagram for uniaxial tension

For $\sigma_{ct} \leqslant 0.9 f_{ctm}$

For $0.9 f_{ctm} < \sigma_{ct} \leqslant f_{ctm}$

$$\sigma_{ct} = f_{ctm} - \frac{0.1 f_{ctm}}{0.00015 - 0.9 f_{ctm}/E_{ci}} (0.00015 - \varepsilon_{ct}) \tag{2.1–24}$$

where

E_{ci} is the tangent modulus of elasticity in (MPa) from eq. (2.1–15)
f_{ctm} is the tensile strength in (MPa) from eq. (2.1–4)
σ_{ct} is the tensile stress in (MPa)
ε_{ct} is the tensile strain.

For a cracked section a bilinear stress–crack opening relation as given in eqs (2.1–25) to (2.1–27) may be used (see Fig. 2.1.5):

$$\sigma_{ct} = f_{ctm}\left(1 - 0.85 \frac{w}{w_1}\right) \quad \text{for } 0.15 f_{ctm} \leqslant \sigma_{ct} \leqslant f_{ctm} \tag{2.1–25}$$

$$\sigma_{ct} = \frac{0.15 f_{ctm}}{w_c - w_1} (w_c - w) \quad \text{for } 0 \leqslant \sigma_{ct} < 0.15 f_{ctm} \tag{2.1–26}$$

$$w_1 = 2 \frac{G_F}{f_{ctm}} - 0.15 w_c \tag{2.1–27a}$$

$$w_c = \alpha_F \frac{G_F}{f_{ctm}} \tag{2.1–27b}$$

where

w is the crack opening (mm)
w_1 is the crack opening (mm) for $\sigma_{ck} = 0.15 f_{ctm}$
w_c is the crack opening (mm) for $\sigma_{ct} = 0$
G_F is the fracture energy (Nmm/mm^2) from eq. (2.1–7)
f_{ctm} is the tensile strength (MPa) from eq. (2.1–4)
α_F is the coefficient as given in Table 2.1.8.

The coefficient α_F depends on the maximum aggregate size $d_{\max}$ as given in Table 2.1.8.

Table 2.1.8. Coefficient α_F to estimate w_c

$d_{\max}$ (mm)	8	16	32
α_F[–]	8	7	5

This constitutive model is one among several acceptable formulations. It agrees well with test data.

The model is based on non-linear elasticity of the finite type, where the secant modulus of elasticity E_{csa} and Poisson's ratio ν_{csa} depend on the actual state and level of stress. The model describes path-independent, reversible, concrete behaviour. It is restricted to monotonically increasing proportional loading.

The post-failure stress–strain behaviour of concrete under multiaxial states of stress is not covered by the formulae presented here, because sufficient experimental data are not available. For further details refer to 'Concrete under multiaxial states of stress—constitutive equations for practical design', CEB Bulletin 156, Lausanne, 1983 and to Ottosen, N., 'A Failure-Criterion for Concrete', Journal Engineering Mechanics Division, ASCE, Vol. 103, EM4, August 1977.

The term β_{sa} is defined as the non-linearity index. It is a measure of the actual level of stress in relation to a failure state. At failure $\beta_{sa} = 1$. For definitions of β_{sa}, refer to Fig. 2.1.6.

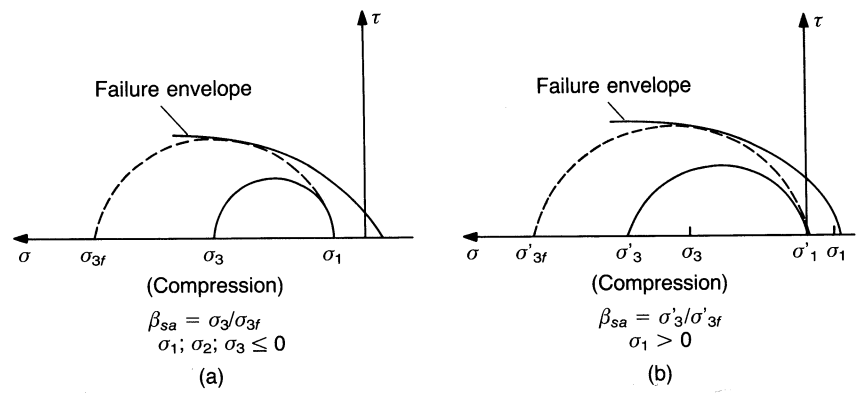


Fig. 2.1.6. Definition of β_{sa}: (a) triaxial compression; (b) one or more principal

2.1.4.4.3. Multiaxial states of stress

The principal strains ε_1, ε_2 and ε_3 of concrete due to a multiaxial state of stress given by the principal stresses σ_1, σ_2 and σ_3 may be estimated from the following constitutive equations

$$\varepsilon_1 = \frac{1}{E_{csa}}[\sigma_1 - \nu_{csa}(\sigma_2 + \sigma_3)] \tag{2.1–28a}$$

$$\varepsilon_2 = \frac{1}{E_{csa}}[\sigma_2 - \nu_{csa}(\sigma_3 + \sigma_1)] \tag{2.1–28b}$$

$$\varepsilon_3 = \frac{1}{E_{csa}}[\sigma_3 - \nu_{csa}(\sigma_1 + \sigma_2)] \tag{2.1–28c}$$

where

E_{csa} is the actual secant modulus of elasticity for different stress levels due to the stress state according to eq. (2.1–29)

ν_{csa} is the actual Poisson's ratio for different stress levels according to eq. (2.1–33).

The actual secant modulus of elasticity may be estimated from eq. (2.1–29)

$$E_{csa} = \frac{E_{ci}}{2} - \beta_{sa}\left(\frac{E_{ci}}{2} - E_{cf}\right) + \left\{\left[\frac{E_{ci}}{2} - \beta_{sa}\left(\frac{E_{ci}}{2} - E_{cf}\right)\right]^2 - E_{cf}^2\beta_{sa}\right\}^{1/2} \tag{2.1–29}$$

where

$$\beta_{sa} = \frac{\sigma_3}{\sigma_{3f}} \quad \text{for } \sigma_1; \sigma_2; \sigma_3 \leqslant 0 \tag{2.1–30a}$$

$$\beta_{sa} = \frac{\sigma'_3}{\sigma'_{3f}} \quad \text{for } \sigma_1 > 0 \tag{2.1–30b}$$

$$E_{cf} = \frac{E_{c1}}{1 + 4[(E_{ci}/E_{c1}) - 1]\zeta} \quad \text{for } \zeta > 0 \tag{2.1–31a}$$

$$E_{cf} = E_{c1} \quad \text{for } \zeta \leqslant 0 \tag{2.1–31b}$$

$$\zeta = (\sqrt{J_{2f}}/f_{cm}) - (1/\sqrt{3}) \tag{2.1–32}$$

For triaxial compression σ_{3f} is obtained by increasing the actual stress σ_3 up to a failure state which may be determined from the failure criterion given in clause 2.1.3.4 (refer to Fig. 2.1.6(a)).

If at least one of the principal stresses is tension, i.e. $\sigma_1 > 0$, concrete behaviour becomes less non-linear. To account for this, the actual stress state is transformed by superimposing a state of hydrostatic compression $-\sigma_1$ resulting in a stress state σ_1'; σ_2'; σ_3' (refer to Fig. 2.1.6(b)).

The actual secant modulus of elasticity E_{csa} decreases as β_{sa} increases. At a failure state, i.e. $\beta_{sa} = 1$, $E_{csa} = E_{cf}$ according to eq. (2.1–31).

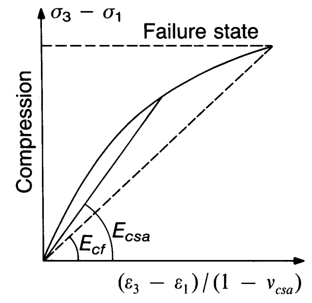


Fig. 2.1.7. Definition of E_{csa} and E_{cf}

where

E_c is the tangent modulus of elasticity for uniaxial compression according to eq. (2.1–15)

$E_{c1} = f_{cm}/|\varepsilon_{c1}|$ is the secant modulus of elasticity at failure for uniaxial compression (refer to Fig. 2.1.2)

E_{cf} is the secant modulus of elasticity at failure for a multiaxial state of stress according to Fig. 2.1.7

σ_3 is the largest principal compressive stress or smallest principal tensile stress

σ_{3f} is the principal stress σ_3 causing failure provided that σ_1 and σ_2 are unchanged (refer to Fig. 2.1.6(a))

$\sigma_3' = \sigma_3 - \sigma_1$

σ_{3f}' is the principal stress σ_3' causing failure provided that $\sigma_1' = 0$ and $\sigma_2' = \sigma_2 - \sigma_1$ are unchanged (refer to Fig. 2.1.6(b))

J_{2f} is the stress deviator J_2 for a failure stress state σ_1; σ_2 and σ_{3f} or $\sigma_1' = 0$; σ_2'; σ_{3f}' respectively.

For triaxial compression ζ in eq. (2.1–32) describes a decrease of E_{cf} with an increase of the deviatoric stress at failure. If at least one of the principal stresses is tension, ζ becomes negative and does not influence E_{cf} (refer to eq. (2.1–31b)).

According to eqs (2.1–33) the actual Poisson's ratio is constant up to $\beta_{sa} = 0.8$ and reaches a final value of 0.36 at failure.

The actual Poisson's ratio ν_{csa} may be estimated from eq. (2.1–33a) or (2.1–33b)

$$\nu_{csa} = \nu_c \text{ if } \beta_{sa} \leqslant 0.8 \qquad (2.1\text{–}33a)$$

$$\nu_{csa} = 0.36 - (0.36 - \nu_c)\sqrt{[1 - (5\beta_{sa} - 4)^2]} \text{ if } \beta_{sa} > 0.8 \qquad (2.1\text{–}33b)$$

where

ν_c is Poisson's ratio as given in clause 2.1.4.3
β_{sa} is the ratio to be taken from eq. (2.1–30).

This method is particularly suitable for numerical analysis. For further details refers to Darwin, D., Pecknold, D.A.W., 'Inelastic model for cyclic biaxial loading of reinforced concrete', Civil Engineering Studies, Structural Research Series, No. 409, University of Illinois, July 1974, and to Chen, W.F., Saleb, A.F., 'Constitutive equations for engineering materials', Vol. 1, p. 397 ff, John Wiley and Sons, 1982.

In FE analysis special attention has to be paid to the fact that element orientation and principal stress orientation do not necessarily coincide and may rotate with increasing stresses.

Stress–strain relations for biaxial states of stress may also be estimated from the following relations (2.1–34) to (2.1–42). They are based on the concept of equivalent uniaxial strain.

The incremental constitutive relations for a plane state of stress of an orthotropic material may be expressed by eq. (2.1–34):

$$\begin{Bmatrix} d\sigma_x \\ d\sigma_y \\ d\tau_{xy} \end{Bmatrix} = \frac{1}{1-\nu^2}\begin{bmatrix} E_x & \nu\sqrt{(E_xE_y)} & 0 \\ \nu\sqrt{(E_xE_y)} & E_y & 0 \\ 0 & 0 & (1-\nu^2)G \end{bmatrix}\begin{Bmatrix} d\varepsilon_x \\ d\varepsilon_y \\ d\gamma_{xy} \end{Bmatrix} \tag{2.1–34}$$

where

$$\nu = \sqrt{(\nu_x\nu_y)} \tag{2.1–35a}$$

and

$$(1-\nu^2)G = \tfrac{1}{4}[E_x + E_y - 2\nu\sqrt{(E_xE_y)}] \tag{2.1–35b}$$

where

σ_x; σ_y: τ_{xy} are the normal stresses and shear stress, respectively, acting in the x-y plane
ε_x; ε_y; γ_{xy} are the normal strains and shear strain, respectively, acting in the x-y plane
E_x; E_y are the tangential moduli of elasticity
G is the shear modulus
ν_x; ν_y is Poisson's ratio for a stress increment $d\sigma_x$; $d\sigma_y$
ν is the equivalent Poisson's ratio.

Note that the shear modulus $G = E/2(1+\nu)$ is assumed to be invariant with respect to the axis orientation.

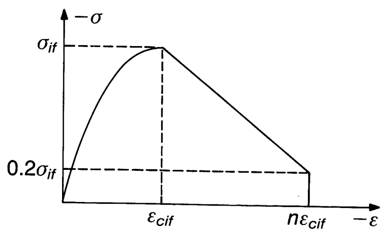
For the direction of principal stresses 1; 2; eq. (2.1–34) simplifies to

$$\begin{Bmatrix} d\sigma_1 \\ d\sigma_2 \end{Bmatrix} = \frac{1}{1-\nu^2}\begin{bmatrix} E_1 & \nu\sqrt{(E_1E_2)} \\ \nu\sqrt{(E_1E_2)} & E_2 \end{bmatrix}\begin{Bmatrix} d\varepsilon_1 \\ d\varepsilon_2 \end{Bmatrix} \tag{2.1–36a}$$

The variations of the tangent moduli of elasticity E_1; E_2 with stress are determined from the equivalent uniaxial strains ε_{iu}

$$d\varepsilon_{iu} = d\sigma_i/E_i \quad (i = 1;\ 2) \tag{2.1–37a}$$

$$\varepsilon_{iu} = \sum_K d\varepsilon_{iu} \quad (K = 1 \ldots \text{stress increments}) \tag{2.1–37b}$$



where

$d\varepsilon_{iu}$ is the equivalent uniaxial strain increment
E_i is the tangent modulus of elasticity at a stress σ_i.

If the strains ε_i in eq. (2.1–36a) are substituted by the equivalent uniaxial strains ε_{iu} the incremental constitutive relations may be expressed by eq. (2.1–36b)

$$\begin{Bmatrix} d\sigma_1 \\ d\sigma_2 \end{Bmatrix} = \begin{bmatrix} E_1 & 0 \\ 0 & E_2 \end{bmatrix} \begin{Bmatrix} d\varepsilon_{1u} \\ d\varepsilon_{2u} \end{Bmatrix} \qquad (2.1\text{–}36b)$$

The equivalent uniaxial strain for compression may be estimated from eq. (2.1–18) by replacing the peak stress $\sigma_c = -f_{cm}$ by the corresponding failure stress σ_{if}, the strain at peak stress ε_{c1} by an equivalent strain ε_{cif} and E_{c1} by $\sigma_{if}/\varepsilon_{cif}$. Relations for ε_{cif} are given in eqs (2.1–39) to (2.1–42).

Biaxial compression and biaxial tension-compression for $\sigma_{3u} < -0.96 f_{cm}$

$$\varepsilon_{c3f} = \varepsilon_{c1}\left(-3\frac{\sigma_{3f}}{f_{cm}} - 2\right) \qquad (2.1\text{–}39a)$$

$$\varepsilon_{c2f} = \varepsilon_{c1}\left(-3\frac{\sigma_{2f}}{f_{cm}} - 2\right) \qquad (2.1\text{–}39b)$$

$$\nu = \text{const.} = 0.2$$

where

ε_{c2f}; ε_{c3f} are the equivalent uniaxial strains at peak stress
$\varepsilon_{c1} = -0.0022$ which is the strain at peak stress $\sigma_c = -f_{cm}$ for uniaxial compression
σ_{3f}; σ_{2f} are the principal stresses at failure from eq. (2.1–12)
ν is the Poisson's ratio to be used in eq. (2.1–34).

Biaxial tension

$$\varepsilon_{cif} = \varepsilon_{ct1} = 0.00015 \qquad (2.1\text{–}40)$$

$$\nu = \text{const.} = 0.2$$

where

ε_{ct1} is the strain at peak stress f_{ctm} for uniaxial tension (Fig. 2.1.5).

Note that by introducing the equivalent uniaxial strains in eq. (2.1–36b) the effect of Poisson's ratio is eliminated and taken into account only in eq. (2.1–34).

For compression, the descending part of the stress-equivalent uniaxial strain relationship may be approximated by a straight line as expressed by eq. (2.1–38) and shown in Fig. 2.1.8.

$$\frac{\sigma_{ci}}{\sigma_{if}} = -\frac{\varepsilon_{ci}}{\varepsilon_{cif}}\frac{0.8}{n-1} + \frac{n-0.2}{n-1} \quad \text{for} \quad |\varepsilon_{cif}| < |\varepsilon_c| < |n\varepsilon_{cif}| \qquad (2.1\text{–}38)$$

The cut-off strain $n\varepsilon_{cif}$ of the descending part of the stress–strain relationship according to Fig. 2.1.8 depends on the strength grade as given in Table 2.1.9.

Table 2.1.9. Coefficient n to describe the cut-off strain

Concrete grade	C20	C40	C60	C80
n	3	2	1.5	1.2

Biaxial tension-compression for $\sigma_{3u} \geqslant -0.96 f_{cm}$

$$\varepsilon_{c3f} = \varepsilon_{c1} \left\{ -1.6 \left(\frac{-\sigma_{3f}}{f_{cm}} \right)^3 + 2.25 \left(\frac{-\sigma_{3f}}{f_{cm}} \right)^2 + 0.35 \left(\frac{-\sigma_{3f}}{f_{cm}} \right) \right\} \quad (2.1\text{–}41a)$$

$$\varepsilon_{cif} = \varepsilon_{ct1} \quad (2.1\text{–}41b)$$

$$\nu(\sigma) = 0.2 + 0.6 \left(\frac{-\sigma_3}{f_{cm}} \right)^4 + 0.4 \left(\frac{\sigma_1}{f_{ctm}} \right)^4 < 0.99 \quad (2.1\text{–}42)$$

Note that ν according to eq. (2.1–42) is valid for stress increments $d\sigma_1$; $d\sigma_3$. The 'secant' value of ν at maximum stress is approx. 0.35 to 0.40.

2.1.5. Stress and strain rate effects — impact

2.1.5.1. Range of applicability

The information given in subsection 2.1.5 is valid for monotonically increasing compressive stresses or strains at a constant range of approximately $1\,\text{MPa/s} < |\dot{\sigma}_c| < 10^7\,\text{MPa/s}$ and $30 \times 10^{-6}\,\text{s}^{-1} < |\dot{\varepsilon}_c| < 3 \times 10^2\,\text{s}^{-1}$, respectively.

For tensile stresses or strains it is valid for $0.1\,\text{MPa/s} < \dot{\sigma}_{ct} < 10^7\,\text{MPa/s}$ and $3 \times 10^{-6}\,\text{s}^{-1} < \dot{\varepsilon}_{ct} < 3 \times 10^2\,\text{s}^{-1}$, respectively.

For further details refer to 'Concrete Structures under Impact and Impulsive Loading' Synthesis Report, CEB Bulletin 187, Lausanne, 1988.

2.1.5.2. Compressive strength

For a given stress rate the compressive strength under high rates of loading may be estimated from eqs (2.1–43):

$$f_{c,imp}/f_{cm} = (\dot{\sigma}_c/\dot{\sigma}_{co})^{\alpha_s} \quad \text{for } |\dot{\sigma}_c| \leqslant 10^6\,\text{MPa/s} \quad (2.1\text{–}43a)$$

$$f_{c,imp}/f_{cm} = \beta_s(\dot{\sigma}_c/\dot{\sigma}_{co})^{1/3} \quad \text{for } |\dot{\sigma}_c| > 10^6\,\text{MPa/s} \quad (2.1\text{–}43b)$$

with

$$\alpha_s = \frac{1}{5 + 9 f_{cm}/f_{cmo}} \quad (2.1\text{–}44a)$$

and

$$\log \beta_s = 6\alpha_s - 2 \quad (2.1\text{–}44b)$$

where

$f_{c,imp}$ is the impact compressive strength,
$\dot{\sigma}_c$ is the stress rate (MPa/s)

$f_{cmo} = 10\,\text{MPa}$
$\dot{\sigma}_{co} = -1\,\text{MPa/s}$.

For a given strain rate the compressive strength may be estimated from eq. (2.1–45):

$$f_{c,imp}/f_{cm} = (\dot{\varepsilon}_c/\dot{\varepsilon}_{co})^{1.026\alpha_s} \quad \text{for } |\dot{\varepsilon}_c| \leqslant 30\,\text{s}^{-1} \qquad (2.1\text{–}45a)$$

$$f_{c,imp}/f_{cm} = \gamma_s(\dot{\varepsilon}_c/\dot{\varepsilon}_{co})^{1/3} \quad \text{for } |\dot{\varepsilon}_c| > 30\,\text{s}^{-1} \qquad (2.1\text{–}45b)$$

with

$$\log \gamma_s = 6.156\alpha_s - 2 \qquad (2.1\text{–}46)$$

where

$\dot{\varepsilon}_c$ is the strain rate (s^{-1})
$\dot{\varepsilon}_{co} = -30 \times 10^{-6}\,\text{s}^{-1}$
α_s is the coefficient from eq. (2.1–44a).

2.1.5.3. Tensile strength and fracture properties

2.1.5.3.1. Tensile strength

For a given stress rate the tensile strength under high rates of loading may be estimated from eqs (2.1–47):

$$f_{ct,imp}/f_{ctm} = (\dot{\sigma}_{ct}/\dot{\sigma}_{cto})^{\delta_s} \quad \text{for } \dot{\sigma}_{ct} \leqslant 10^6\,\text{MPa/s} \qquad (2.1\text{–}47a)$$

$$f_{ct,imp}/f_{ctm} = \lambda_s(\dot{\sigma}_{ct}/\dot{\sigma}_{cto})^{1/3} \quad \text{for } \dot{\sigma}_{ct} > 10^6\,\text{MPa/s} \qquad (2.1\text{–}47b)$$

with

$$\delta_s = \frac{1}{10 + 6 f_{cm}/f_{cmo}} \qquad (2.1\text{–}48a)$$

$$\log \lambda_s = 7\delta_s - 7/3 \qquad (2.1\text{–}48b)$$

where

$f_{ct,imp}$ is the impact tensile strength
$\dot{\sigma}_{ct}$ is the stress rate (MPa/s)
f_{ctm} is the tensile strength from eq. (2.1–4)
$\dot{\sigma}_{cto} = 0.1\,\text{MPa/s}$
$f_{cmo} = 10\,\text{MPa}$.

For a given strain rate the tensile strength under high rates of loading may be estimated from eqs (2.1–49):

$$f_{ct,imp}/f_{ctm} = (\dot{\varepsilon}_{ct}/\dot{\varepsilon}_{cto})^{1.016\delta_s} \quad \text{for } \dot{\varepsilon}_{ct} \leqslant 30\,\text{s}^{-1} \qquad (2.1\text{–}49a)$$

$$f_{ct,imp}/f_{ctm} = \beta_s(\dot{\varepsilon}_{ct}/\dot{\varepsilon}_{cto})^{1/3} \quad \text{for } \dot{\varepsilon}_{ct} > 30\,\text{s}^{-1} \qquad (2.1\text{–}49b)$$

with

$$\log \beta_s = 7.112\delta_s - 2.33 \qquad (2.1\text{–}50)$$

where

$\dot{\varepsilon}_{ct}$ is the strain rate (s^{-1})
$\dot{\varepsilon}_{cto} = 3 \times 10^{-6} s^{-1}$.

2.1.5.3.2. Fracture energy

The information available regarding the effect of stress or strain rate on fracture energy is too incomplete to be included in this Model Code.

2.1.5.4. Modulus of elasticity

Eqs (2.1–51) are valid for all grades of concrete.

The effect of stress and strain rate on modulus of elasticity may be estimated from eqs (2.1–51)

$$E_{c,imp}/E_{ci} = (\dot{\sigma}_c/\dot{\sigma}_{co})^{0.025} \qquad (2.1\text{–}51a)$$

$$E_{c,imp}/E_{ci} = (\dot{\varepsilon}_c/\dot{\varepsilon}_{co})^{0.026} \qquad (2.1\text{–}51b)$$

where

$E_{c,imp}$ is the impact modulus of elasticity
E_{ci} is the modulus of elasticity of concrete from eqs (2.1–15) and (2.1–16)
$\dot{\sigma}_c$ is the stress rate (MPa/s)
$\dot{\varepsilon}_c$ is the strain rate (s^{-1})
$\dot{\sigma}_{co} = -1.0$ MPa/s and $\dot{\varepsilon}_{co} = -30 \times 10^{-6} s^{-1}$ for compression
$\dot{\sigma}_{cto} = 0.1$ MPa/s and $\dot{\varepsilon}_{cto} = 3 \times 10^{-6} s^{-1}$ for tension.

The effects of high stress and strain rates on the strains at maximum stress in tension and compression, respectively, may be estimated from eq. (2.1–52):

$$\varepsilon_{c1,imp}/\varepsilon_{c1} = (\dot{\sigma}_c/\dot{\sigma}_{co})^{0.02} = (\dot{\varepsilon}_c/\dot{\varepsilon}_{co})^{0.02} \qquad (2.1\text{–}52)$$

with

$\dot{\sigma}_{co} = -1$ MPa/s and $\dot{\varepsilon}_{co} = -30 \times 10^{-6}\,\mathrm{s}^{-1}$ for compression
$\dot{\sigma}_{cto} = 0.1$ MPa/s and $\dot{\varepsilon}_{cto} = 3 \times 10^{-6}\,\mathrm{s}^{-1}$ for tension

where

$\varepsilon_{c1,imp}$ is the impact strain at maximum load
ε_{c1} is the strain at maximum load for static loading from clauses 2.1.4.4.1 and 2.1.4.4.2 for compression and tension, respectively.

The development of tensile strength with time is strongly influenced by curing and drying conditions as well as by the dimensions of the structural members. As a first approximation it may be assumed that for a duration of moist curing $t_s \leqslant 7$ days and a concrete age $t > 28$ days the development of tensile strength is similar to that of compressive strength, i.e. eq. (2.1–4) is independent of concrete age for $t \geqslant 28$ days. For a concrete age $t < 28$ days residual stresses may cause a temporary decrease of the tensile strength.

In cases where the development of tensile strength with time is important it is recommended to carry out experiments taking into account exposure conditions and dimensions of the structural member.

2.1.5.5. Stress–strain diagrams

There is little information regarding the effect of high stress or strain rates on the shape of the stress–strain diagrams.

As an approximation, for monotonically increasing compressive stresses or strains up to the peak stress eq. (2.1–18) may be used together with eqs (2.1–43) and (2.1–45) for the peak stress $f_{c,imp}$, eq. (2.1–51) for the modulus of elasticity, $E_{c,imp}$, and eq. (2.1–52) for the strain at maximum stress, $\varepsilon_{c1,imp}$.

No information is available for the strain-softening region.

2.1.6. Time effects

2.1.6.1. Development of strength with time

The compressive strength of concrete at an age t depends on the type of cement, temperature and curing conditions. For a mean temperature of 20°C and curing in accordance with ISO 2736/2 the relative compressive strength of concrete at various ages $f_{cm}(t)$ may be estimated from eqs (2.1–53) and (2.1–54). To take into account the effect of temperature during curing the actual concrete age should be adjusted according to eq. (2.1–87).

$$f_{cm}(t) = \beta_{cc}(t) f_{cm} \qquad (2.1\text{–}53)$$

with

$$\beta_{cc}(t) = \exp\left\{ s \left[1 - \left(\frac{28}{t/t_1} \right)^{1/2} \right] \right\} \qquad (2.1\text{–}54)$$

where

$f_{cm}(t)$ is the mean concrete compressive strength at an age of t days
f_{cm} is the mean compressive strength after 28 days according to eq. (2.1–1)
$\beta_{cc}(t)$ is a coefficient which depends on the age of concrete t
t is the age of concrete (days) adjusted according to eq. (2.1–87)
$t_1 = 1$ day
s is a coefficient which depends on the type of cement (for cement classification, refer to Appendix d, clause d.4.2.1): $s = 0.20$ for rapid hardening high strength cements RS, 0.25 for normal and rapid hardening cements N and R, and 0.38 for slowly hardening cements SL.

Due to the counteracting effects of the parameters influencing the strength under sustained loads, $f_{cm,sus}(t, t_0)$ passes through a minimum. The duration of loading for which this minimum occurs depends on the age of loading and is referred to as the critical period $(t - t_0)_{crit}$. For an age at loading of 28 days, a concrete made of normal cement, type N, $(t - t_0)_{crit} = 2.8$ (days), $f_{c,sus\,min} = 0.78 f_{cm}$. It is generally referred to as sustained load strength of concrete.

There is insufficient experimental basis to give information on the tensile strength of concrete under high sustained tensile stresses.

2.1.6.2. Strength under sustained loads

When subjected to sustained high compressive stresses the compressive strength of concrete decreases with time under load. This strength reduction is counteracted by a strength increase due to continued hydration. The combined effect of sustained stresses and of continued hydration is given by eqs (2.1–55) and (2.1–56)

$$f_{cm,sus}(t, t_0) = f_{cm}\beta_{cc}(t)\beta_{c,sus}(t, t_0) \qquad (2.1\text{–}55)$$

with

$$\beta_{c,sus}(t, t_0) = 0.96 - 0.12\left\{\ln\left[72\left(\frac{t - t_0}{t_1}\right)\right]\right\}^{1/4} \qquad (2.1\text{–}56)$$

where

$f_{cm,sus}(t, t_0)$ is the mean compressive strength of concrete at time t when subjected to a high sustained compressive stress at an age at loading $t_0 < t$
$\beta_{cc}(t)$ is a coefficient according to eq. (2.1–54)
$\beta_{c,sus}(t, t_0)$ is a coefficient which depends on the time under high sustained loads $t - t_0$ (days). The coefficient describes the decrease of strength with time under load and is defined for $(t - t_0) > 0.015$ days (=20 min)
t_0 is the age of the concrete at loading
$t - t_0$ is the time under high sustained loads (days)
$t_1 = 1$ day.

2.1.6.3. Development of modulus of elasticity with time

The modulus of elasticity of concrete at an age $t \neq 28$ days may be estimated from eq. (2.1–57):

$$E_{ci}(t) = \beta_E(t)E_{ci} \qquad (2.1\text{–}57)$$

with

$$\beta_E(t) = [\beta_{cc}(t)]^{0.5} \qquad (2.1\text{–}58)$$

where

$E_{ci}(t)$ is the modulus of elasticity at an age of t days
E_{ci} is the modulus of elasticity at an age of 28 days, from eq. (2.1–16)
$\beta_E(t)$ is a coefficient which depends on the age of concrete, t (days)

2.1.6.4. Creep and shrinkage

2.1.6.4.1. Definitions

The total strain at time t, $\varepsilon_c(t)$, of a concrete member uniaxially loaded at time t_0 with a constant stress $\sigma_c(t_0)$ may be expressed as follows

$$\varepsilon_c(t) = \varepsilon_{ci}(t_0) + \varepsilon_{cc}(t) + \varepsilon_{cs}(t) + \varepsilon_{cT}(t) \quad (2.1\text{–}59)$$

$$= \varepsilon_{c\sigma}(t) + \varepsilon_{cn}(t) \quad (2.1\text{–}60)$$

where

$\varepsilon_{ci}(t_0)$ is the initial strain at loading
$\varepsilon_{cc}(t)$ is the creep strain at time $t > t_0$
$\varepsilon_{cs}(t)$ is the shrinkage strain
$\varepsilon_{cT}(t)$ is the thermal strain
$\varepsilon_{c\sigma}(t)$ is the stress dependent strain: $\varepsilon_{c\sigma}(t) = \varepsilon_{ci}(t_0) + \varepsilon_{cc}(t)$
$\varepsilon_{cn}(t)$ is the stress independent strain: $\varepsilon_{cn}(t) = \varepsilon_{cs}(t) + \varepsilon_{cT}(t)$.

The distinction between creep and shrinkage is conventional. Normally the delayed strains of loaded or unloaded concrete should be considered as two aspects of a single physical phenomenon.

Also, separation of initial strain and creep strain is a matter of convention. In structural analysis, the total load dependent strain as given by the creep function (refer to clause 2.1.6.4.3) is of importance. The initial and creep strain components are defined consistently, so that their sum results in the correct load dependent strain.

For the prediction of the creep function the initial strain $\varepsilon_{ci}(t)$ is based on the tangent modulus of elasticity as defined in eqs (2.1–15) and (2.1–57).

2.1.6.4.2. Range of applicability

The prediction model for creep and shrinkage given below predicts the mean behaviour of a concrete cross-section.

Unless special provisions are given the model is valid for ordinary structural concrete (12 MPa $< f_{ck} \leqslant$ 80 MPa) subjected to a compressive stress $|\sigma_c| < 0.4 f_{cm}(t_0)$ at an age of loading t_0 and exposed to mean relative humidities in the range of 40 to 100% at mean temperatures from 5°C to 30°C.

It is accepted that the scope of the model also extends to concrete in tension, though the relations given in the following are directed towards the prediction of creep of concrete subjected to compressive stresses.

The model does not predict local rheological properties within the cross-section of a concrete member such as variations due to internal stresses, moisture states or the effects of local cracking.

The prediction model is not applicable to

- concrete subjected to extreme temperatures, high (e.g. nuclear reactors) or low (e.g. LNG-tanks)
- very dry climatic conditions (average relative humidity RH $<$ 40%)
- structural lightweight aggregate concrete.

The effect of temperature variations during hardening can be taken into account in accordance with eq. (2.1–87). The effect of 0°C $< T <$ 80°C is dealt with in subsection 2.1.8.

2.1.6.4.3. Creep

(a) Assumptions and related basic equations

Within the range of service stresses $|\sigma_c| < 0.4 f_{cm}(t_0)$, creep is assumed to be linearly related to stress.

For a constant stress applied at time t_0 this leads to

$$\varepsilon_{cc}(t, t_0) = \frac{\sigma_c(t_0)}{E_{ci}} \phi(t, t_0) \quad (2.1\text{–}61)$$

Here, concrete is considered as an ageing linear visco-elastic material. In reality, creep is a non-linear phenomenon. The non-linearity with respect to creep inducing stress may be observed in creep experiments at a constant stress, particularly if the stress exceeds $0.4 f_{cm}(t_o)$, as well as in experiments with a variable stress history even below stresses of $0.4 f_{cm}(t_o)$.

In this section a so-called product formulation for the prediction of creep has been used, i.e. creep after a given duration of loading can be predicted from the product of a notional creep coefficient which depends on the age of concrete at loading and a function describing the development of creep with time. As an alternative, creep may also be described by a summation formulation as the sum of delayed elastic and of viscous strains. Advantages and disadvantages of both approaches as well as an alternative prediction model based on a summation formulation are given in: 'Evaluation of the time dependent behaviour of concrete', CEB Bulletin 199, Lausanne, 1990.

where

$\phi(t, t_0)$ is the creep coefficient
E_{ci} is the modulus of elasticity at the age of 28 days according to (eq. (2.1–15) or (2.1–16)).

The stress dependent strain $\varepsilon_{c\sigma}(t, t_0)$ may be expressed as

$$\varepsilon_{c\sigma}(t, t_0) \;=\; \sigma_c(t_0)\left[\frac{1}{E_c(t_0)} + \frac{\phi(t, t_0)}{E_{ci}}\right] = \sigma_c(t_0)J(t, t_0) \qquad (2.1\text{–}62)$$

where

$J(t, t_0)$ is the creep function or creep compliance, representing the total stress dependent strain per unit stress
$E_c(t_0)$ is the modulus of elasticity at the time of loading t_0 according to eq. (2.1–57); hence $1/E_c(t_0)$ represents the initial strain per unit stress at loading.

The application of the principle of superposition is consistent with respect to the assumption of linearity. However, due to the actual non-linear behaviour of concrete some prediction errors are inevitable when linear superposition is applied to creep of concrete under variable stress, particularly for unloading or decreasing strains, respectively. For linear creep prediction models, the error depends on the type of model which is underlying the creep prediction (refer to CEB Bulletin 177).

The structural effects of time-dependent behaviour of concrete are dealt with in detail in section 5.8 of this Model Code and in CEB-Manual on 'Structural Effects of Time-dependent Behaviour of Concrete', CEB Bulletin 142/142 bis, Lausanne, 1984.

For variable stresses or strains, the principle of superposition is assumed to be valid.

On the basis of the assumptions and definitions given above, the constitutive equation for concrete may be written as

$$\varepsilon_c(t) \;=\; \sigma_c(t_0)J(t, t_0) + \int_{t_0}^{t} J(t, \tau)\,\frac{\partial\sigma_c(\tau)}{\partial\tau}\,d\tau + \varepsilon_{cn}(t) \qquad (2.1\text{–}63)$$

The relations to calculate the creep coefficient are empirical. They were calibrated on the basis of laboratory tests (creep in compression) on structural concretes.

In this prediction model only those parameters are taken into account which normally are known to the designer, i.e. characteristic compressive strength, dimensions of the member, mean relative humidity to which the member is exposed, age at loading, duration of loading and type of cement. It should be pointed out, however, that creep of concrete does not depend on its compressive strength or age at loading per se, but rather on its composition and degree of hydration; creep of concrete decreases with

(b) Creep coefficient

The creep coefficient may be calculated from

$$\phi(t, t_0) \;=\; \phi_0\beta_c(t - t_0) \qquad (2.1\text{–}64)$$

where

ϕ_0 is the notional creep coefficient eq. (2.1–65)
β_c is the coefficient to describe the development of creep with time after loading eq. (2.1–70)
t is the age of concrete (days) at the moment considered
t_0 is the age of concrete at loading (days), adjusted according to

decreasing water/cement ratio, decreasing cement content and increasing degree of hydration.

Due to the inherent scatter of creep and shrinkage deformations, the errors of the model and the general uncertainty caused by randomness of material properties and environment, a deformation prediction may result in a considerable prediction error. After short durations of loading or drying the prediction error is higher than after long durations of loading and drying. Based on a computerized data bank of laboratory test results a mean coefficient of variation for the predicted creep function $V_c = 20\%$ has been estimated. Assuming a normal distribution this corresponds to a 10 and 5 percent cut-off, respectively, on the lower and the upper side of the mean value of

$$\phi_{0.10} = 0.74\phi;\ \phi_{0.05} = 0.66\phi$$
$$\phi_{0.90} = 1.26\phi;\ \phi_{0.95} = 1.34\phi$$

The prediction error should be taken into account in a probabilistic approach where appropriate.

The notional creep coefficient may be estimated from

$$\phi_0 = \phi_{RH}\beta(f_{cm})\beta(t_0) \qquad (2.1\text{–}65)$$

with

$$\phi_{RH} = 1 + \frac{1 - RH/RH_o}{0.46(h/h_o)^{1/3}} \qquad (2.1\text{–}66)$$

$$\beta(f_{cm}) = \frac{5.3}{(f_{cm}/f_{cmo})^{0.5}} \qquad (2.1\text{–}67)$$

$$\beta(t_0) = \frac{1}{0.1 + (t_0/t_1)^{0.2}} \qquad (2.1\text{–}68)$$

where

$$h = 2A_c/u \qquad (2.1\text{–}69)$$

f_{cm} is the mean compressive strength of concrete at the age of 28 days (MPa) according to eq. (2.1–1)
$f_{cmo} = 10\,\text{MPa}$
RH is the relative humidity of the ambient environment (%)
$RH_o = 100\%$
h is the notational size of member (mm), where A_c is the cross-section and u is the perimeter of the member in contact with the atmosphere
$h_o = 100\,\text{mm}$
$t_1 = 1$ day.

It is not known whether creep approaches a finite value. Nevertheless, the hyperbolic time function given in eq. (2.1–70) approaches an asymptotic value for $t \to \infty$. Evaluations on the basis of test results indicate that eq. (2.1–70) is a reasonably good approximation for the time development of creep up to 70 years of loading under the conditions indicated in Table 2.1.10. From experimental observations of creep up to 30 years one may conclude that the increase of creep from 70 years up to 150 years of duration of loading will not exceed 5% of the creep after 70 years.

The development of creep with time is given by

$$\beta_c(t - t_0) = \left[\frac{(t - t_0)/t_1}{\beta_H + (t - t_0)/t_1}\right]^{0.3} \qquad (2.1\text{–}70)$$

with

$$\beta_H = 150\left\{1 + \left(1.2\frac{RH}{RH_o}\right)^{18}\right\}\frac{h}{h_o} + 250 \leqslant 1500 \qquad (2.1\text{–}71)$$

where

$t_1 = 1$ day
$RH_o = 100\%$
$h_o = 100\,\text{mm}$.

In cases where a lower level of accuracy is sufficient, the values given in Table 2.1.10 can be accepted as representative values for the creep coefficient after 70 years of loading of a normal weight ordinary structural concrete with a characteristic compressive strength between 20 and 50 MPa. These 70 year values may be taken as final creep coefficients.

Table 2.1.10. Creep coefficient ϕ (70y, t_0) of an ordinary structural concrete after 70 years of loading

Age at loading t_0 (days)	Dry atmospheric conditions (indoors) (RH = 50%)			Humid atmospheric conditions (out of doors) (RH = 80%)		
	Notional size $2A_c/u$ (mm)					
	50	150	600	50	150	600
1	5.8	4.8	3.9	3.8	3.4	3.0
7	4.1	3.3	2.7	2.7	2.4	2.1
28	3.1	2.6	2.1	2.0	1.8	1.6
90	2.5	2.1	1.7	1.6	1.5	1.3
365	1.9	1.6	1.3	1.2	1.1	1.0

The data given in Table 2.1.10 apply for a mean temperature of the concrete between 10°C and 20°C. Seasonal variations of temperature between −20°C and +40°C can be accepted. The same is true for variations in relative humidity around the mean values given in Table 2.1.10.

(c) Effect of type of cement and curing temperature

The effect of type of cement on the creep coefficient of concrete may be taken into account by modifying the age at loading t_0 according to eq. (2.1–72):

$$t_0 = t_{0,T}\left[\frac{9}{2 + (t_{0,T}/t_{1,T})^{1.2}} + 1\right]^{\alpha} \geqslant 0.5 \text{ days} \qquad (2.1\text{–}72)$$

where

$t_{0,T}$ is the age of concrete at loading (days) adjusted according to eq. (2.1–87)

$t_{1,T}$ = 1 day

For classification of different types of cement refer to Appendix d, clause d.4.2.1.

Different types of cement result in different degrees of hydration. Creep of concrete depends on the degree of hydration reached at a given age rather than on the age of concrete. Therefore, the effect of type of cement is taken into account by modifying the age at loading such that for a given modified age the degree of hydration is approximately independent of the type of cement. The value for t_0 according to eq. (2.1–72) has to be used in eq. (2.1–68). The duration of loading $t - t_0$ used in eq. (2.1–70) is the actual time under load.

The creep behaviour of concrete with blended cements may as a first approximation be calculated with the formulae given here. However, larger prediction errors may be expected.

α is the power which depends on the type of cement;

α = −1 for slowly hardening cements SL, 0 for normal or rapid hardening cements N and R, and 1 for rapid hardening high strength cements RS.

The main reasons for the non-linear behaviour are micro-cracking due to shrinkage or high loads and stress-induced ageing under load.

Eq. (2.1–73a) represents a simplification in so far as it does not take into account the observation that non-linearity decreases with increasing duration of loading and with decreasing change of moisture content during loading.

It should be noted that delayed elastic strains upon total unloading are linear functions of stress up to stress levels of $|\sigma_c| = 0.6 f_{cm}(t_0)$.

For mass concrete and at very high relative humidities, the coefficient α_σ may be as low as $\alpha_\sigma = 0.5$.

(*d*) *Effect of high stresses*

For stress levels in the range of $0.4 f_{cm}(t_0) < |\sigma_c| < 0.6 f_{cm}(t_0)$ the non-linearity of creep may be taken into account using eqs (2.1–73)

$$\phi_{0,k} = \phi_0 \exp[\alpha_\sigma(k_\sigma - 0.4)] \quad \text{for } 0.4 < k_\sigma \leqslant 0.6 \tag{2.1–73a}$$

$$\phi_{0,k} = \phi_0 \quad \text{for } k_\sigma \leqslant 0.4 \tag{2.1–73b}$$

where

$\phi_{0,k}$ is the non-linear notional creep coefficient, which replaces ϕ_0 in eq. (2.1–64)

$k_\sigma = |\sigma_c|/f_{cm}(t_0)$ which is the stress–strength ratio

$\alpha_\sigma = 1.5$.

For curing periods of concrete members $t_s < 14$ days at normal ambient temperatures, the duration of moist curing does not significantly affect shrinkage. Hence, this parameter as well as the effect of curing temperature is not taken into account.

In eqs (2.1–74) and (2.1–79) the actual duration of drying $(t - t_s)$ has to be used. It is not affected by possible adjustments of t_0 or t_s according to eqs (2.1–72) and (2.1–87).

Similar to creep, shrinkage does not depend on concrete compressive strength per se. Shrinkage decreases with decreasing water/cement ratio and decreasing cement content.

A mean coefficient of variation of predicted shrinkage has been estimated on the basis of a computerized data bank, resulting in $V_s = 35\%$. The corresponding 10 and 5 percent cut-off values are

2.1.6.4.4. Shrinkage

The total shrinkage or swelling strains $\varepsilon_{cs}(t, t_s)$ may be calculated from

$$\varepsilon_{cs}(t, t_s) = \varepsilon_{cso}\beta_s(t - t_s) \tag{2.1–74}$$

where

ε_{cso} is the notional shrinkage coefficient (eq. (2.1–75))

β_s is the coefficient to describe the development of shrinkage with time (eq. (2.1–79))

t is the age of concrete (days)

t_s is the age of concrete (days) at the beginning of shrinkage or swelling.

The notional shrinkage coefficient may be obtained from

$$\varepsilon_{cso} = \varepsilon_s(f_{cm})\beta_{RH} \tag{2.1–75}$$

$$\varepsilon_{cs0.10} = 0.55\varepsilon_{cs};\ \varepsilon_{cs0.05} = 0.42\varepsilon_{cs}$$
$$\varepsilon_{cs0.90} = 1.45\varepsilon_{cs};\ \varepsilon_{cs0.95} = 1.58\varepsilon_{cs}$$

In cases where a lower level of accuracy is sufficient, the values given in Table 2.1.11 can be accepted as representative values for shrinkage of a normal weight ordinary structural concrete with a characteristic strength between 20 and 50 MPa after 70 years of drying. Usually these values may be taken as final shrinkage values.

Table 2.1.11. Shrinkage values $\varepsilon_{cs,70y} \times 10^3$ for an ordinary structural concrete after a duration of drying of 70 years

Dry atmospheric conditions (inside) (RH = 50%)			Humid atmospheric conditions (outside) (RH = 80%)		
Notional size $2A_c/u$ (mm)					
50	150	600	50	150	600
−0.57	−0.56	−0.47	−0.32	−0.31	−0.26

Though shrinkage reaches a final value, little information exists on the shrinkage strains of large members after long durations of drying. Therefore, the values calculated using eq. (2.1–79) for $2A_c/u$ = 500 mm, and the values given in Table 2.1.11 for shrinkage of members with a notional size of $2A_c/u$ = 600 mm, respectively, are uncertain and may overestimate the actual shrinkage strains after 70 years of drying.

with

$$\varepsilon_s(f_{cm}) = [160 + 10\beta_{sc}(9 - f_{cm}/f_{cmo})] \times 10^{-6} \tag{2.1–76}$$

where

f_{cm} is the mean compressive strength of concrete at the age of 28 days (MPa)
f_{cmo} = 10 MPa
β_{sc} is a coefficient which depends on the type of cement: $\beta_{Sc} = 4$ for slowly hardening cements SL, $\beta_{Sc} = 5$ for normal or rapid hardening cements N and R, and $\beta_{Sc} = 8$ for rapid hardening high strength cements RS,

$$\beta_{RH} = -1.55\beta_{sRH} \quad \text{for } 40\% \leqslant RH < 99\%$$
$$\beta_{RH} = +0.25 \quad \text{for } RH \geqslant 99\% \tag{2.1–77}$$

where

$$\beta_{sRH} = 1 - \left(\frac{RH}{RH_o}\right)^3 \tag{2.1–78}$$

with

RH is the relative humidity of the ambient atmosphere (%)
RH_o = 100%.

The development of shrinkage with time is given by

$$\beta_s(t - t_s) = \left[\frac{(t - t_s)/t_1}{350(h/h_o)^2 + (t - t_s)/t_1}\right]^{0.5} \tag{2.1–79}$$

where

h is defined in eq. (2.1–69)
t_1 = 1 day
h_o = 100 mm.

2.1.7. Fatigue

2.1.7.1. Fatigue strength

For a constant stress amplitude the number N of cycles causing fatigue failure of plain concrete may be estimated from eqs (2.1–80) to (2.1–85). They are valid for pure compression, compression–tension and pure tension, respectively.

Fatigue tests exhibit a large scatter in the number of cycles to failure. Therefore, often probabilistic procedures are applied in evaluating fatigue behaviour of concrete. For further details refer to 'Fatigue of concrete structures'. State-of-the-Art Report, CEB Bulletin 188, Lausanne, 1988.

Pure compression

For $S_{c,min} > 0.8$, the S–N relations for $S_{c,min} = 0.8$ are valid.

For $0 \leqslant S_{c,min} \leqslant 0.8$, eqs (2.1–80, 81 and 82) apply

$$\log N_1 = (12 + 16 S_{c,min} + 8 S_{c,min}^2)(1 - S_{c,max}) \qquad (2.1\text{–}80)$$

$$\log N_2 = 0.2 \log N_1 (\log N_1 - 1) \qquad (2.1\text{–}81)$$

$$\log N_3 = \log N_2 (0.3 - 0.375 \cdot S_{c,min}) / \Delta S_c \qquad (2.1\text{–}82)$$

The relations (2.1–80) to (2.1–82) are valid for concrete tested under sealed conditions. They are also valid for large concrete sections of low permeability. Thin concrete sections which are allowed to dry may exhibit higher fatigue strengths, whereas permeable concrete immersed in water may have a lower fatigue strength than expressed by these relations.

(a) If $\log N_1 \leqslant 6$, then $\log N = \log N_1$.
(b) If $\log N_1 > 6$ and $\Delta S_c \geqslant 0.3 - 0.375 \cdot S_{c,min}$, then $\log N = \log N_2$.
(c) If $\log N_1 > 6$ and $\Delta S_c < 0.3 - 0.375 \cdot S_{c,min}$, then $\log N = \log N_3$.

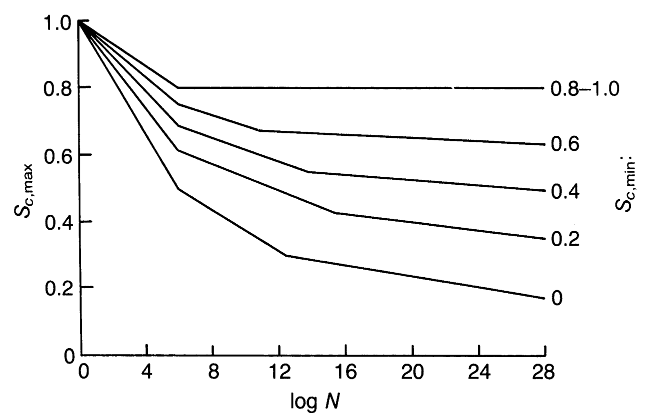


Fig. 2.1.9. S–N relations according to eqs (2.1–80) to (2.1–82)

with

$$S_{c,max} = |\sigma_{c,max}| / f_{ck,fat}$$

$$S_{c,min} = |\sigma_{c,min}| / f_{ck,fat}$$

$$\Delta S_c = |S_{c,max}| - |S_{c,min}|$$

The fatigue reference compressive strength $f_{ck,fat}$ has been introduced to take into account the increasing fatigue sensitivity of concrete with increasing compressive strength.

If eq. (2.1–84) is applied it may be assumed that the concrete always fails in compression.

The fatigue lives given by these equations correspond to a probability of failure $p = 5\%$ in a log-normal distribution for any given maximum stress. If limited data are available for an estimate of fatigue lives the evaluation of the 5% defective of fatigue life should be done at a confidence level of 75%.

Eqs (2.1–80) to (2.1–85) are applicable for stress levels $S_{c,\max}$ and $S_{ct,\max} < 0.9$ and for frequencies $f > 0.1$ cycle/min. For higher stress levels and lower frequencies, i.e. low cycle fatigue, lower values of $\log N$ than predicted by eqs (2.1–80) to (2.1–85) may be expected. For further details refer to CEB Bulletin 188.

A value of $\beta_{c,sus}(t, t_0) = 0.85$ has been chosen to take account of actual frequencies of loading which are in most practical cases significantly lower than those applied in experiments.

The fatigue reference compressive strength $f_{ck,fat}$ may be estimated from eq. (2.1–83)

$$f_{ck,fat} = \beta_{cc}(t)\beta_{c,sus}(t, t_0) f_{ck}(1 - f_{ck}/25 f_{cko}) \qquad (2.1\text{–}83)$$

Compression–tension with $\sigma_{ct,\max} \leqslant 0.026|\sigma_{c,\max}|$

$$\log N = 9\,(1 - S_{c,\max}) \qquad (2.1\text{–}84)$$

Pure tension and tension-compression with $\sigma_{ct,\max} > 0.026|\sigma_{c,\max}|$

$$\log N = 12\,(1 - S_{ct,\max}) \qquad (2.1\text{–}85)$$

with

$$S_{ct,\max} = \sigma_{ct,\max}/f_{ctk,\min}$$

where

N is the number of cycles to failure
$S_{c,\max}$ is the maximum compressive stress level
$S_{c,\min}$ is the minimum compressive stress level
$S_{ct,\max}$ is the maximum tensile stress level
ΔS_c is the stress range
$\sigma_{c,\max}$ is the maximum compressive stress
$\sigma_{c,\min}$ is the minimum compressive stress
$\sigma_{ct,\max}$ is the maximum tensile stress
f_{ck} is the characteristic compressive strength from Table 2.1.1
$f_{ck,fat}$ is the fatigue reference compressive strength from eq. (2.1–83)
$f_{cko} = 10\,\text{MPa}$
$f_{ctk,\min}$ is the minimum characteristic tensile strength from eq. (2.1–2)
$\beta_{cc}(t)$ is a coefficient which depends on the age of concrete at the beginning of fatigue loading, to be taken from clause 2.1.6.1, eq. (2.1–54)
$\beta_{c,sus}(t, t_0)$ is a coefficient which takes into account the effect of high mean stresses during loading (refer to clause 2.1.6.2, eq. (2.1–56): for fatigue loading it may be taken as $\beta_{c,sus}(t, t_0) = 0.85$.

To estimate the fatigue life for a spectrum of load levels the Palmgren-Miner summation may be applied. Fatigue failure occurs if $D = 1$.

$$D = \sum_i \frac{n_{Si}}{n_{Ri}}$$

where

D is the fatigue damage

n_{Si} is the number of acting stress cycles at a given stress level and stress range

n_{Ri} is the number of cycles causing failure at the same stress level and stress range according to eqs (2.1–80) to (2.1–85).

2.1.7.2. Fatigue strains

For maximum compressive stresses $|\sigma_{c,\max}| < 0.6 f_{ck}$ and a mean stress $(|\sigma_{c,\max}| + |\sigma_{c,\min}|)/2 < 0.5 f_{ck}$ the strain at maximum stress due to repeated loads of a given frequency f may be estimated from eq. (2.1–86):

$$\varepsilon_{cf}(n) = \frac{\sigma_{c,\max}}{E_{ci}(t_0)} + \frac{\sigma_{c,\max} + \sigma_{c,\min}}{2E_{ci}} \phi(t, t_0) \qquad (2.1\text{–}86)$$

where

ε_{cf} is the strain at maximum stress due to repeated loads
$\sigma_{c,\max}$ is the maximum compressive stress
$\sigma_{c,\min}$ is the minimum compressive stress
E_{ci} is the modulus of elasticity of concrete at a concrete age of 28 days according to eq. (2.1–15)
$E_{ci}(t_0)$ is the modulus of elasticity of concrete at a concrete age t_0 according to (2.1–57)
$\phi(t, t_0)$ is the creep coefficient according to eq. (2.1–64)
t_0 is the age of concrete at the beginning of repeated loading (days)
t is the age of concrete at the moment considered (days).

In eq. (2.1–86) it is assumed that creep due to repeated loading is equal to creep under a constant stress $(\sigma_{c,\max} + \sigma_{c,\min})/2$ acting during a time $(t - t_0) = \frac{1}{1440} n/f$ = duration of repeated loading (days), where

n is the number of cycles applied at a frequency f
f is the frequency of repeated loading ($\min^{-1}$).

Therefore, eq. (2.1–86) gives only a rough estimate of the creep strains due to repeated loads. It does not take into account variations of E_c due to repeated loads as well as of tertiary creep which develops prior to fatigue failure. For further details refer to CEB Bulletin 188.

2.1.8. Temperature effects

2.1.8.1. Range of application

The information given in the preceding sections is valid for a mean temperature taking into account seasonal variations, between approx. −20°C and +40°C. In the following section the effect of substantial deviations from a mean concrete temperature of 20°C for the range of approximately 0°C to +80°C is dealt with.

The data given in this section are limited to a maximum temperature +80°C because the information available on concrete properties for $T > 80$°C is too complex for a Code type formulation, particularly with regard to the effects of type of aggregates and transient or steady moisture states. For such conditions experimental studies, using the particular concrete composition, are recommended.

2.1.8.2. Maturity

The effect of elevated or reduced temperatures on the maturity of concrete may be taken into account by adjusting the concrete age according to eq. (2.1–87):

$$t_T = \sum_{i=1}^{n} \Delta t_i \exp\left[13.65 - \frac{4000}{273 + T(\Delta t_i)/T_0}\right] \quad (2.1\text{–}87)$$

where

t_T is the temperature adjusted concrete age which replaces t in the corresponding equations
Δt_i is the number of days where a temperature T prevails
$T(\Delta t_i)$ is the temperature (°C) during the time period Δt_i
$T_0 = 1°C$.

The activation energy for concrete hydration is influenced by the type of cement and additions. Eq. (2.1–87) is valid for concretes made of Portland cements or cements containing only low amounts of components other than Portland cement clinker (CE I and CE II according to Appendix d, clause d.4.2.1).

2.1.8.3. Thermal expansion

Thermal expansion of concrete may be calculated from eq. (2.1–88)

$$\varepsilon_{cT} = \alpha_T \Delta T \quad (2.1\text{–}88)$$

where

ε_{cT} is the thermal strain
ΔT is the change of temperature (K)
α_T is the coefficient of thermal expansion (K^{-1}).

For the purpose of structural analysis the coefficient of thermal expansion may be taken as $\alpha_T = 10 \times 10^{-6} K^{-1}$.

The coefficient of thermal expansion depends on the type of aggregates and on the moisture state of the concrete. Thus it may vary between approx. $6 \times 10^{-6} K^{-1}$ and $15 \times 10^{-6} K^{-1}$. A value of $10 \times 10^{-6} K^{-1}$ holds true, e.g. for concrete made of quartzitic aggregates.

2.1.8.4. Compressive strength

The effect of temperature at the time of testing in the range of $0°C < T < 80°C$ on the compressive strength of concrete without exchange of moisture (e.g. mass concrete) may be calculated from eq. (2.1–89)

$$f_{cm}(T) = f_{cm}(1.06 - 0.003T/T_0) \quad (2.1\text{–}89)$$

where

$f_{cm}(T)$ is the compressive strength at the temperature T
f_{cm} is the compressive strength at the temperature 20°C from eq. (2.1–1)
T is the temperature in (°C)
$T_0 = 1°C$.

In cases where moisture exchange takes place, the effect of temperature on compressive strength depends on size and shape of the member. As a first approximation the effect of temperature in the range of $0°C < T < 80°C$ on compressive strength may be neglected since the reduction in strength due to a temperature increase is offset by an increase in strength due to drying.

2.1.8.5. Tensile strength and fracture properties

In the range of $0°C < T < 80°C$ the uniaxial tensile strength f_{ct} and the tensile splitting strength $f_{ct,sp}$ are not significantly affected by temperature at the time of testing.

Eq. (2.1–90) may be used to estimate the effect of elevated or reduced temperatures at the time of testing on flexural strength $f_{ct,fl}$:

$$f_{ct,fl}(T) = f_{ct,fl}(1.1 - 0.005T/T_0) \tag{2.1–90}$$

where

$f_{ct,fl}(T)$ is the flexural strength at the temperature T
$f_{ct,fl}$ is the flexural strength at the temperature 20°C from eq. (2.1–6)
T is the temperature in (°C)
$T_0 = 1°C$.

The effect of temperature at testing on tensile strength of concrete strongly depends on the moisture state of the concrete and on temperature gradients. At elevated temperatures where moisture exchange takes place but no temperature gradient is expected, the effect of temperature may be neglected because a strength reduction due to increased temperature is offset by a strength increase due to drying. Only insufficient information is available on the influence of temperature gradients on the tensile strength.

Fracture energy G_F is strongly affected by temperature and moisture content at the time of testing. The effect of temperature on G_F may be estimated from eqs (2.1–91a) and (2.1–91b):

dry concrete: $$G_F(T) = G_F(1.06 - 0.003T/T_0) \tag{2.1–91a}$$

mass concrete: $$G_F(T) = G_F(1.12 - 0.006T/T_0) \tag{2.1–91b}$$

where

$G_F(T)$ is the fracture energy at a temperature T
G_F is the fracture energy at a temperature of 20°C from eq. (2.1–7)
T is the temperature (°C)
$T_0 = 1°C$.

2.1.8.6. Modulus of elasticity

The effect of elevated or reduced temperatures at the time of testing on the modulus of elasticity of concrete, at an age of 28 days without exchange of moisture, may be estimated from eq. (2.1–92)

$$E_{ci}(T) = E_{ci}(1.06 - 0.003T/T_0) \tag{2.1–92}$$

where

$E_{ci}(T)$ is the modulus of elasticity at the temperature T
E_{ci} is the modulus of elasticity at the temperature 20°C from eq. (2.1–15)
T is the temperature (°C)
$T_0 = 1°C$.

If moisture exchange takes place the effect of temperature on the modulus of elasticity is generally more pronounced than expressed by eq. (2.1–92).

2.1.8.7. Creep and shrinkage

2.1.8.7.1. Creep

The effect of temperature prior to loading may be taken into account using eq. (2.1–87).

Eqs (2.1–93) to (2.1–96) describe the effect of a constant temperature differing from 20°C while the concrete is under load.

The effect of temperature on the time development of creep is taken into account using $\beta_{H,T}$ from eq. 2.1–93

$$\beta_{H,T} = \beta_H \beta_T \qquad (2.1\text{–}93)$$

with

$$\beta_T = \exp[1500/(273 + T/T_0) - 5.12] \qquad (2.1\text{–}94)$$

where

$\beta_{H,T}$ is a temperature dependent coefficient replacing β_H in eq. (2.1–70)
β_H is a coefficient according to eq. (2.1–71)
$T_0 = 1°C$.

The effect of temperature on the creep coefficient is taken into account using eqs (2.1–95) and (2.1–96)

$$\phi_{RH,T} = \phi_T + (\phi_{RH} - 1)\phi_T^{1.2} \qquad (2.1\text{–}95)$$

with

$$\phi_T = \exp[0.015(T/T_0 - 20)] \qquad (2.1\text{–}96)$$

where

$\phi_{RH,T}$ is a temperature dependent coefficient which replaces ϕ_{RH} in eq. (2.1–65)
ϕ_{RH} is a coefficient according to eq. (2.1–66)
$T_0 = 1°C$.

For an increase of temperature while the structural member is under load, creep may be estimated from eq. (2.1–97)

$$\phi(t, t_0, T) = \phi_0 \beta_c(t - t_0) + \Delta\phi_{T,trans} \qquad (2.1\text{–}97)$$

with

$$\Delta\phi_{T,trans} = 0.0004(T/T_0 - 20)^2 \qquad (2.1\text{–}98)$$

The relations to predict the effect of temperature up to 80°C on creep given in this section are only rough estimates. For a more accurate prediction considerably more sophisticated models are required which take into account the moisture state of the concrete at the time of loading and distinguish between basic creep and drying creep. Neglecting these parameters the relations given in this section are generally more accurate for thick concrete members with little change in moisture content than for thin members where significant changes in moisture content occur, particularly at elevated temperatures.

where

ϕ_0 is the notional creep coefficient according to eq. (2.1–65) and temperature adjusted according to eq. (2.1–95)

$\beta_c(t - t_0)$ is a coefficient to describe the development of creep with time after loading according to eq. (2.1–70) and temperature adjusted according to eqs (2.1–93) and (2.1–94)

$\Delta\phi_{T,trans}$ is the transient thermal creep coefficient which occurs at the time of the temperature increase

$T_0 = 1°C$.

2.1.8.7.2. Shrinkage

The effect of elevated temperatures on shrinkage is influenced considerably by the moisture content of the concrete prior to heating and the moisture loss after an increase of temperature. Eqs (2.1–99) to (2.1–101) represent shrinkage of a concrete after prolonged curing ($t_s > 14d$) or predrying.

Eqs (2.1–99) to (2.1–101) describe the effect of a constant temperature differing from 20°C while the concrete is drying.

The effect of temperature on the time development of shrinkage is taken into account using $\alpha_{sT}(T)$ from eq. (2.1–99):

$$\alpha_{sT}(T) = 350\left(\frac{h}{h_o}\right)^2 \exp[-0.06(T/T_0 - 20)] \qquad (2.1\text{–}99)$$

where

$\alpha_{sT}(T)$ is a temperature dependent coefficient replacing the product $350(h/h_o)^2$ in eq. (2.1–79)

$h_o = 100\,\text{mm}$

$T_0 = 1°C$.

The effect of temperature on the notional shrinkage coefficient is taken into account using eq. (2.1–100)

$$\beta_{RH,T} = \beta_{RH}\beta_{sT} \qquad (2.1\text{–}100)$$

with

$$\beta_{sT} = 1 + \left(\frac{8}{103 - 100RH/RH_o}\right)\left(\frac{T/T_0 - 20}{40}\right) \qquad (2.1\text{–}101)$$

where

$\beta_{RH,T}$ is a temperature dependent coefficient which replaces β_{RH} in eq. (2.1–75)

β_{RH} is a coefficient according to eq. (2.1–77)

$RH_o = 100\%$

$T_0 = 1°C$.

2.1.9. Transport of liquids and gases in hardened concrete

Liquids, gases or ions may be transported in hardened concrete by various transport mechanisms:

- permeation
- diffusion
- capillary suction
- mixed modes of transport mechanisms.

Transport characteristics are difficult to predict since they may vary by several orders of magnitude depending on concrete composition, type of materials, age, curing and moisture content of the concrete. Therefore, when a more accurate prediction of transport characteristics is required, they should be determined experimentally. For further details refer to RILEM-TC-116 PCD, State-of-the-Art Report: Performance Criteria for Concrete Durability. (To be published.)

2.1.9.1. Permeation

Permeation is the flow of liquids, e.g. water, or of gases, e.g. air, caused by a pressure head.

2.1.9.1.1. Water permeability

The transport of water is generally described by Darcy's law, eq. (2.1–102)

$$V = K_w \frac{A}{l} \Delta h_w t \qquad (2.1\text{–}102)$$

where

V is the volume of water (m^3) flowing during time t
Δh_w is the hydraulic head (m)
A is the penetrated area (m^2)
t is the time (s)
l is the thickness (m)
K_w is the coefficient of water permeability for water flow (m/s).

For mature concrete the coefficient of water permeability may be estimated roughly from the characteristic strength of concrete according to eq. (2.1–103):

$$\log(K_w/K_{wo}) = -0.7 f_{ck}/f_{cko} \qquad (2.1\text{–}103)$$

where

K_w is the coefficient of water permeability (m/s)
f_{ck} the characteristic strength (MPa)
$K_{wo} = 10^{-10}$ m/s
$f_{cko} = 10$ MPa.

In concrete the flow of water does not only occur in the capillary pores of the paste but also through internal microcracks as well as along the porous interfaces between the matrix and coarse aggregates. These effects compensate the low permeability of dense aggregates, so that in general the permeability of concrete is equal to or larger than the permeability of the hydrated cement paste matrix.

The flow of water in the hydrated cement paste depends on the presence of interconnected capillary pores which are mainly determined by the water/cement ratio of the mix and the degree of hydration of the cement. Despite a low water/cement ratio, insufficient curing, which may result in a low degree of hydration especially in the near surface region, may lead to a high permeability, whereas a high degree of hydration results in a low permeability even for higher water/cement ratios.

2.1.9.1.2. Gas permeability

For a stratified laminar flow the volume of gas flowing through a porous material is given by eq. (2.1–104):

$$V = K_g \frac{A}{l} \frac{p_1 - p_2}{\eta} p_m \frac{1}{p} t \qquad (2.1\text{–}104)$$

with

V is the volume of gas (m^3) flowing during time t
K_g is the coefficient of gas permeability (m^2)
A is the penetrated area (m^2)
l is the thickness (m) of the penetrated section
$p_1 - p_2$ is the pressure difference (N/m^2)
p_m is the mean pressure $= (p_1 + p_2)/2$ (N/m^2)
η is the viscosity of gas (Ns/m^2)
p is the local pressure, at which V is observed (Ns/m^2)
t is the time (s).

Similar to the flow of water, gases may pass through the pore system and microcracks of concrete under the influence of an external pressure. The coefficient of permeability K_g (m^2) in eq. (2.1–104) represents a constant material parameter. Therefore, the viscosity η of the gas flowing, as well as the pressure level p, have to be considered in the calculation of the volume of gas V.

If only one type of gas is considered η is normally taken as unity. Then K_g represents the specific permeability for the gas considered, and is given in (m/s).

If also the influence of the pressure level p_m is neglected, the volume of gas flowing can be calculated from

$$V = \bar{K}_g \frac{A}{l} \frac{p_1 - p_2}{p} t \qquad (2.1\text{–}105)$$

where

$\bar{K}_g$ is the coefficient of gas permeability (m^2/s).

As a rough estimate, K_g for air or oxygen may be determined from the characteristic compressive strength of concrete f_{ck} from eq. (2.1–106)

$$\log(K_g/K_{go}) = -0.5 f_{ck}/f_{cko} \qquad (2.1\text{–}106)$$

where

$K_{go} = 10^{-14}\,m^2$
$f_{cko} = 10\,MPa$
K_g is the coefficient of gas permeability (m^2).

Aside from the pore structure of the concrete, the moisture content exerts an essential influence on its gas permeability. Eq. (2.1–106) is valid for a relative pore humidity of the concrete of less than about 65%. With increasing relative humidity of the concrete K_g may be reduced by a factor up to 10^{-3}.

2.1.9.2. Diffusion

Gases, liquids and dissolved substances are transported due to a constant concentration gradient according to Fick's first law of diffusion

$$Q = D \frac{c_1 - c_2}{l} A t \qquad (2.1\text{–}107)$$

where

Q is the amount of substance transported (g)

In most cases transient diffusion phenomena occur, i.e. the amount of substance diffusing varies with location x and time t. From Fick's first law of diffusion the balance for a volume element penetrated is derived as the second law of diffusion, which describes the change in concentration for an element with time according to eq. (2.1–108) which is valid for one-dimensional flow

$$\frac{\partial c}{\partial t} = D \frac{\partial^2 c}{\partial x^2} \qquad (2.1\text{–}108)$$

$c_1 - c_2$ is the difference in concentration (g/m^3)
l is the thickness of the penetrated section (m)
A is the penetrated area (m^2)
t is the time (s)
D is the diffusion coefficient (m^2/s).

In cases where the diffusing substance becomes immobile, such as in the case of diffusion of chloride ions, eq. (2.1–108) has to be expanded

$$\frac{\partial c}{\partial t} = D\frac{\partial^2 c}{\partial x^2} + s \qquad (2.1\text{–}109a)$$

where s = sink, i.e. amount of transported substance which becomes immobile.

Frequently, the diffusion of ions is described by eq. (2.1–109b):

$$\frac{\partial c_{free}}{\partial t} = D_{eff}\frac{\partial^2 c_{free}}{\partial x^2} \qquad (2.1\text{–}109b)$$

where c_{free} = concentration of free ions, D_{eff} = effective diffusion coefficient. If some of the ions become immobile this is taken into account by an adjustment of the diffusion coefficient. Therefore, D_{eff} in eq. (2.1–109b) is not a constant but varies with time of exposure.

2.1.9.2.1. Diffusion of water

The transport of water in the vapour phase can be described by Fick's first law of diffusion introducing a gradient of the relative pore humidity as the driving force. The diffusion coefficient D is a non-linear function of the local relative pore humidity H. The volume of water flowing is given by eq. (2.1–110)

$$V = D(H)\frac{dH}{dx}At \qquad (2.1\text{–}110)$$

where

V is the volume of transported water (m^3)
$D(H)$ is the diffusion coefficient (m^2/s) at relative pore humidity H
dH/dx is the gradient in relative pore humidity (m^{-1})
A is the penetrated area (m^2)
t is time (s).

For isothermal conditions the diffusion coefficient can be expressed as a function of the relative pore humidity $0 < H < 1$ according to eq. (2.1–111)

$$D(H) = D_1\left[\alpha + \frac{1-\alpha}{1 + [(1-H)/(1-H_c)]^n}\right] \qquad (2.1\text{–}111)$$

The transport of water vapour in the pore system of concrete involves different transport mechanisms and driving forces, therefore $D \neq$ const. In most cases diffusion theory is applied to describe moisture migration. As driving force the local moisture concentration c (g/m^3) may be considered. However, a more convenient approach is the definition of a relative pore humidity $0 < H < 1$ which is correlated with the moisture concentration c by sorption isotherms.

For transient phenomena, such as drying of a concrete cross-section, the balance equation (2.1–108) is transformed to

$$\frac{\partial H}{\partial t} = \frac{\partial}{\partial x}\left(D(H)\frac{\partial H}{\partial x}\right) \qquad (2.1\text{–}112)$$

Eq. (2.1–111) is taken from Bazant, Z.P., Najjar, L.J., 'Drying of concrete as a non-linear diffusion problem', Cement and Concrete Research, Vol. 1, pp. 461–473, 1971.

where

D_1 is the maximum of $D(H)$ for $H = 1$ (m²/s)
D_0 is the minimum of $D(H)$ for $H = 0$ (m²/s)
$\alpha = D_0/D_1$
H_c is the relative pore humidity at $D(H) = 0.5D_1$
n is an exponent
H is the relative pore humidity.

The following approximate values may be assumed

$\alpha = 0.05$
$H_c = 0.80$
$n = 15.$

D_1 may be estimated from eq. (2.1–113)

$$D_1 = \frac{D_{1,o}}{f_{ck}/f_{cko}} \tag{2.1–113}$$

with

$D_{1,o} = 1 \times 10^{-9}$ (m²/s)
$f_{cko} = 10$ MPa.

2.1.9.2.2. Diffusion of gases

The diffusion of gases such as air, oxygen (O_2) or carbon dioxide (CO_2) is primarily controlled by the moisture content of the concrete. For intermediate moisture contents the diffusion coefficient for carbon dioxide or oxygen is in the range of

$$10^{-7} < D < 10^{-10}\,\mathrm{m^2/s}$$

The diffusion coefficient for carbon dioxide D_{CO2} through carbonated concrete may be estimated from eq. (2.1–114)

$$\log(D_{CO2}/D_{CO2,O}) = -0.5 f_{ck}/f_{cko} \tag{2.1–114}$$

with

$D_{CO2O} = 10^{-6.5}$ (m²/s)
$f_{cko} = 10$ MPa.

Eq. (2.1–114) is valid for concrete stored in a constant environment of approximately 20°C, 65% relative humidity. For concrete exposed to a natural environment, particularly to rain, the diffusion coefficient is substantially lower than estimated from eq. (2.1–114).

Based on eqs (2.1–109) and (2.1–114) the progress of carbonation of a concrete under controlled conditions may be estimated from eq. (2.1–115):

$$d_c^2 = 2D_{CO2}\frac{C_a}{C_c}t \qquad (2.1\text{–}115)$$

where

d_c is the depth of carbonation at time t (m)
D_{CO2} is the diffusion coefficient of CO_2 through carbonated concrete (m^2/s) (from eq. (2.1–114))
C_a is the concentration of CO_2 in the air (g/m^3)
C_c is the amount of CO_2 required for complete carbonation of a unit volume of concrete (g/m^3).

For normal weight concrete made of Portland cement and exposed to a standard environment, C_a/C_c may be taken as 8×10^{-6}.

It should be kept in mind, however, that in particular the relative humidity of the surrounding atmosphere as well as the properties of a particular concrete have a strong influence on D_{CO2} so that eq. (2.1–115) cannot give a reliable estimate of the progress of carbonation of a structure in service.

2.1.9.2.3. Diffusion of chloride ions

The diffusion coefficients of dissolved substances increase with increasing moisture content of the concrete. For chloride ions the effective diffusion coefficient as defined in eq. (2.1–109b) is in the range of

$D_{Cl^-} = 1$ to $10 \times 10^{-12}\,m^2/s$ for concretes made of Portland cement
$D_{Cl^-} = 0.3$ to $5 \times 10^{-12}\,m^2/s$ for concretes made of Portland blast furnace slag-cements.

The prediction of the transport of chloride ions into concrete is very complex because chlorides penetrating into concrete may be transported not only by diffusion but also by capillary suction of a salt solution. In addition, the external chloride concentration is variable, and parts of the chloride ions intruded become immobile due to chemical reaction or time dependent physical adsorption. The amount of chlorides combined depends on the type of cement used and must be in equilibrium with the concentration of chlorides dissolved in the pore water. Only the dissolved chlorides take part in the diffusion process. In carbonated concrete all chlorides are dissolved in the pore water.

2.1.9.3. Capillary suction

Liquids, particularly water, may be transported into concrete by capillary suction or absorption. Water absorption may be expressed by eq. (2.1–116)

$$w = w_1(t/t_1)^n = M_w t^n \qquad (2.1\text{–}116)$$

where

w is the water absorbed per unit area at time t (m^3/m^2)
w_1 is the water absorbed at a given time t_1
t is the duration of water absorption (s)
$n = 0.5$
$M_w = W_1/t_1^n$ is the coefficient of water absorption ($m/s^{0.5}$).

Similar to water permeability, capillary suction is strongly influenced by the moisture content of the concrete. As the pore humidity of the concrete increases the rate of water absorption and thus M_w decrease.

For a uniform pore humidity and no substantial microstructural variations within a concrete section exposed to capillary suction, the exponent n in eq. (2.1–116) may be taken as $n = 0.5$. If the moisture distribution is non-uniform, $n < 0.5$.

For a rough estimate the logarithm of the coefficient of water absortion for a given concrete grade may be determined from eq. (2.1–117):

$$\log(M_w/M_{wo}) \;-\; 0.2 f_{ck}/f_{cko} \qquad (2.1\text{–}117)$$

where

$M_{wo} = 10^{-4}$ ($m/s^{0.5}$)
$f_{cko} = 10$ MPa.

Eq. (2.1–117) is valid for a uniform pore humidity of the concrete of approximately 65%. The coefficient of water absorption, depends not only on the moisture state of the concrete, but also on microstructural parameters, so that predictions solely based on a concrete grade are rather uncertain.

2.2. REINFORCING STEEL

2.2.1. General

Products used as reinforcing steel may be

- bars
- wires
- coiled rods
- welded fabric.

Reinforcing steel is characterized by

- geometry
  - size
  - surface characteristics
- mechanical properties
  - strength and yield stress
  - ductility
  - fatigue behaviour
  - behaviour at extreme temperature

The methods of production, the methods of testing and of certification of conformity are as defined in relevant ISO or CEN Standards or RILEM Recommendations.

- technological properties
  - — bond
  - — bendability
  - — weldability
  - — thermal expansion.

The mechanical and technological properties of reinforcement for structures are defined by Product Standards, and are generally secured by certification schemes and certificates of compliance.

Types of reinforcement not covered by approval documents, can be used after it has been shown that they meet the specified requirements.

Mechanical devices for splicing are dealt with in clause 9.1.2.4.

2.2.2. Classification

Reinforcing steels are normally classified on the basis of their

- size
- characteristic yield stress, which defines the grade
- ductility
- surface characteristics and bond properties
- weldability.

Each product should be clearly identifiable with respect to this classification.

The simultaneous use of steels of various types on the same site is allowed only on condition that no confusion between the types is possible during the construction.

It should be possible to distinguish clearly between

- plain bars of various grades or of various ductility classes
- high bond bars of various grades or of various ductility classes
- reinforcement that is weldable and that which is not.

2.2.3. Geometry

2.2.3.1. Size

For quality control purposes and design calculations, the mechanical properties of a product are referred to the nominal cross-sectional area.

The difference between actual and nominal cross-sectional area should not exceed the limiting values specified in relevant standards.

The actual section is determined by weighing a given length of bar, assuming a density of $7850\,kg/m^3$.

The nominal diameter is defined as the diameter of a plain circular cylinder of the same weight per unit length as the bar.

For welded fabric the following applies

- twin bars are allowed in one direction only
- adequate stiffness of the fabric should be ensured either by a limitation of the maximum spacing of the bars, or by introducing a minimum ratio between the diameter of the transverse bars and the diameter of

2.2.3.2. Surface characteristics

Three shapes or surface characteristics are defined

- plain
- indented
- ribbed.

Ribbed bars or wires are considered as high bond if they satisfy the conditions and requirements imposed by the relevant standards or by the approval documents.

Plain bars do not satisfy these conditions.

For indented wires, reference should be made to relevant standards or technical documents.

Plain smooth wires (cold drawn wires) should not be used for reinforced concrete, except as non-structural reinforcement (spacers etc.) or in the form of welded fabric.

2.2.4. Mechanical properties

The mechanical properties are defined on the basis of standard tests.

The standard tests are defined in relevant ISO and CEN Standards and RILEM Recommendations.

2.2.4.1. Tensile properties

The characteristic values of

- the tensile strength (f_t)
- the yield stress (f_y)
- the total elongation at maximum load (ε_u)

respectively, are denoted f_{tk}, f_{yk} and ε_{uk}.

The requirements apply to the product in the condition in which it is delivered. In the case of coiled rods, the requirements apply to the material after straightening.

The value of f_{yk} should correspond to a 0.2% offset in the characteristic $\sigma - \varepsilon$ diagram.

For steels totally or partially cold-worked by means of axial tension, it will generally be the case that

$$f_{yc} \neq f_{yt}$$

where f_{yc} and f_{yt} are actual yield stresses for compression and tension respectively. The value of f_{yc} to be used in a calculation should therefore be stipulated in the approval documents.

2.2.4.2. Steel grades

The steel grade denotes the value of the specified characteristic yield stress in MPa. The Model Code contemplates reinforcing steel up to Grade 500.

Grades higher than 500 require further consideration concerning the validity of the given rules.

2.2.4.3. Stress–strain diagram

Indicative stress–strain diagrams of reinforcing steel in tension are represented in Fig. 2.2.1.

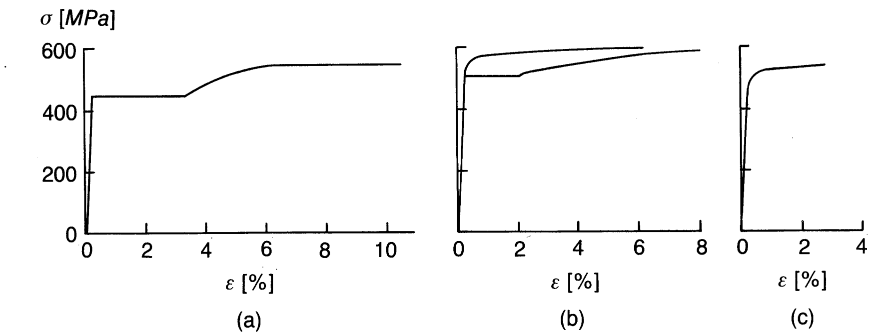


Fig. 2.2.1. Stress–strain relationships of reinforcing steel: (a) hot-rolled bars; heat-treated bars; micro-alloyed bars; (b) low-carbon, heat-treated bars; cold-worked bars; (c) cold-worked wires

Due to the diversity and evolution of the manufacturing processes for bars and wires, various stress–strain diagrams may be encountered.

As a simplification, actual stress–strain diagrams can in calculations be replaced by an idealized characteristic diagram according to Fig. 2.2.2, assuming a modulus of elasticity E_s equal to 200 GPa.

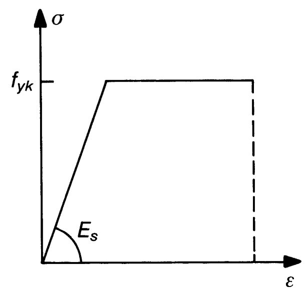


Fig. 2.2.2. Idealized stress–strain diagram

The actual diagram for a particular steel may be used if it is duly verified by the producer.

For high strength steels, the $\sigma - \varepsilon$ diagram is non-symmetrical in compression and in tension.

Some cold-worked steels have a lower modulus of elasticity in compression than in tension. The difference is not important in practice.

2.2.4.4. Ductility

Three ductility classes are defined for design purposes.

These classes are defined by minimum specified values for the characteristic value of the ratio f_t/f_y and the characteristic elongation at maximum load ε_{uk}, as follows.

Class A: $(f_t/f_y)_k \geqslant 1.08$ and $\varepsilon_{uk} \geqslant 5\%$
Class B: $(f_t/f_y)_k \geqslant 1.05$ and $\varepsilon_{uk} \geqslant 2.5\%$
Class S: $(f_t/f_y)_k \geqslant 1.15$ and $\varepsilon_{uk} \geqslant 6\%$.

Adequate ductility is necessary whether or not moment redistribution is taken into account in the calculations.

The characteristic value of the ratio f_t/f_y corresponds to the 5% fractile of the relation between actual tensile strength and actual yield stress.

Class S should be used where high ductility of the structure is required (e.g. in seismic regions).

In seismic design an additional requirement for Class S can be introduced.

2.2.4.5. Fatigue behaviour

The fatigue behaviour of reinforcing steel is described in Table 6.7.1.

Fatigue behaviour depends on factors such as bar size, rib geometry, bending of bars and welded connections, thus making it difficult to give generalized S-N curves. More information can be found in CEB Bulletin d'Information No. 188 'Fatigue of Concrete Structures'.

2.2.5. Technological properties

2.2.5.1. Bond properties

(a) Bars

For ribbed and for some indented products, the bond properties are quantified by means of the projected rib factors.

Ribbed products, having a projected rib factor satisfying the minimum requirements given by the relevant standards, may be assumed to be high bond bars.

Bars not satisfying these requirements should be treated as plain bars with respect to bond. For indented products, which cannot be considered as high bond bars, reference should be made to relevant standards or technical documents.

Poor straightening of ribbed bars and wires from coils can significantly reduce the projected rib factor and thus the bond properties of the straightened bars or wires.

(b) Welded fabric

Where welded joints are taken into account for calculation of the anchorage, each welded joint shall be capable of withstanding a shearing force not less than $0.3A_s f_{yk}$, where A_s denotes the nominal cross-sectional area of the anchored wire.

2.2.5.2. Bendability

The requirements concerning the bendability are specified in relevant standards.

Reinforcing bars should not be bent to a radius less than that used in the relevant rebend test specified in the standards.

2.2.5.3. Weldability

The requirements concerning the weldability are specified in relevant standards.

Depending on the type of reinforcement used, the methods for welding may be restricted.

2.2.5.4. Coefficient of thermal expansion

Within the temperature range from $-20°C$ to $180°C$ the coefficient of thermal expansion of steel may be taken as $10 \times 10^{-6}/°C$.

2.3. PRESTRESSING STEEL

2.3.1. General

Steels for prestressing are delivered as

- wires
- strands
- bars.

The methods of production, the method of testing and of certification of conformity are defined in relevant ISO or CEN Standards or RILEM Recommendations.

Prestressing steel is characterized by

- geometry
  - — size
  - — surface characteristics
- mechanical properties
  - — tensile strength and 0.1% proof-stress
  - — modulus of elasticity
  - — ductility
  - — relaxation
  - — fatigue behaviour
  - — behaviour at extreme temperatures
- technological properties
  - — surface conditions
  - — corrosion resistance
  - — thermal expansion.

The mechanical and technological properties of prestressing steels are defined by standards and are secured by certification schemes and certificates of conformity.

2.3.2. Classification

The classification of prestressing steel is based on values of

- the characteristic tensile strength, which defines the grade
- the characteristics 0.1% proof-stress
- the relaxation class.

Characteristic values are defined in clause 2.3.4.1.

Grade denotes the characteristic tensile strength in MPa, rounded off in tens.

Each product shall be clearly identifiable with respect to this

Each coil of wire or strand or each quantity of bars shall carry a label giving at least

(*a*) the producer's name
(*b*) the product: wire, strand or bar
(*c*) the letters FeP followed by the grade
(*d*) the nominal dimensions (see 2.3.3.1)
(*e*) the surface characteristics (see 2.3.3.2)
(*f*) the relaxation class (see 2.3.4.5).

2.3.3. Geometry

2.3.3.1. Size

For quality control purposes and design calculations, the mechanical properties of a product are referred to the nominal cross-sectional area.

Details of dimensions and configuration along with the accepted tolerances, should be indicated in standards or technical approval documents.
For the test procedures, see document RILEM RPC 10.

2.3.3.2. Surface characteristics

Three shapes or surface characteristics are defined

- smooth
- indented
- ribbed.

2.3.4. Mechanical properties

The mechanical properties are defined on the basis of standard tests.

See document RILEM RPC 9.
The standard tests are defined in relevant ISO and CEN Standards and RILEM Recommendations.

2.3.4.1. Tensile properties

The characteristic values of

- the tensile strength (f_{pt})
- the 0.1% proof-stress ($f_{p0.1}$)
- the total elongation at maximum load (ε_{pu})

respectively designated f_{ptk}, $f_{p0.1k}$ and ε_{puk}, are specified corresponding to the 5% fractile.
The condition $f_{p0.1k} \geqslant 0.80 f_{ptk}$ should be fulfilled.

The characteristic tensile strength is derived from the characteristic failure load.

$0.9 f_{ptk}$ is generally assumed to be a good estimate for $f_{p0.2}$, for design purposes it may even be used for $f_{p0.1k}$, see Fig. 2.3.2.
For convenience in presentation $f_{p0.1k}$ or $f_{p0.2k}$ as relevant for the type of steel used is taken as f_{pyk}.

2.3.4.2. Modulus of elasticity

In case more precise information is not available, the modulus of elasticity of prestressing steel may be taken as

- 205 GPa for wires and bars
- 195 GPa for strands.

2.3.4.3. Force–strain diagrams

Indicative curves are given in Fig. 2.3.1(a) for wires and Fig. 2.3.1(b) for strands.

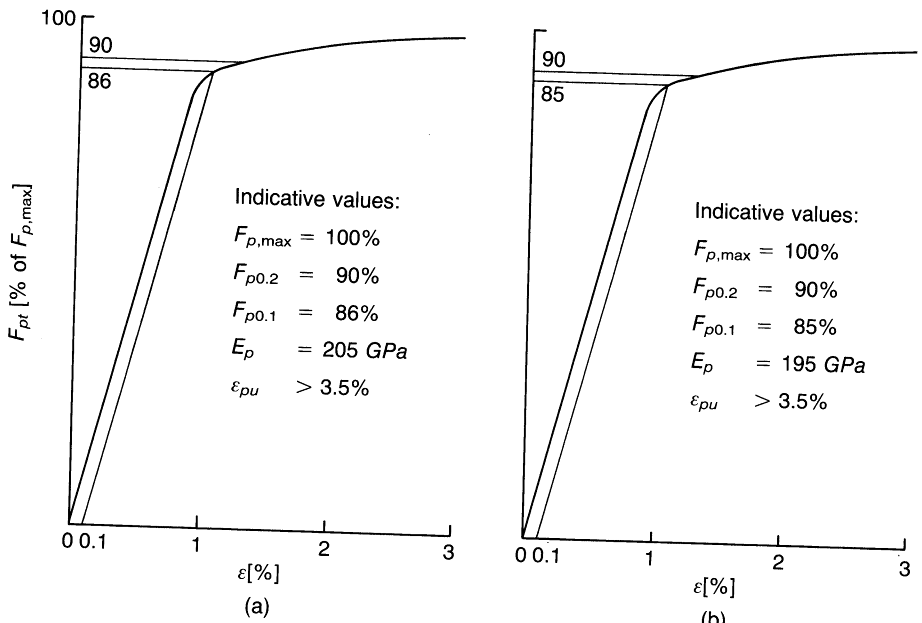


Fig. 2.3.1. Force–strain diagram: (a) for cold-drawn

For calculation purposes the actual stress–strain diagram (corresponding to $f_{pt} = f_{ptk}$) can be replaced by a simplified schematic diagram.

However, it may be necessary in some design situations (see subsections 6.2.4 and 6.2.5) to use the actual stress–strain diagram (duly factored) instead of the idealized one.

Figure 2.3.2 shows a simplified bi-linear diagram. It is valid for temperatures from −20°C to 100°C.

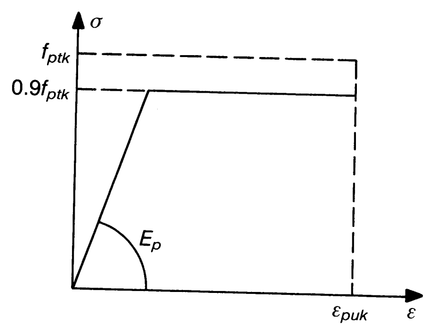


Fig. 2.3.2. Schematic stress–strain diagram

2.3.4.4. Ductility

(*a*) *Reverse bending behaviour*
The minimum number of reverse bends should be

- for smooth wires: 4
- for indented wires: 3.

The minimum number of reverse bends is determined according to ISO 7801.

(*b*) *Behaviour under multiaxial stresses*
Prestressing steels should have an adequate behaviour under multiaxial stresses.

Tests on strands should be carried out according to a test method developed by FIP (see 'Deflected tensile test', FIP Notes 1987/1).
According to this test method, the behaviour may be assumed adequate if the value of 'D' (defined in the document mentioned above) does not exceed a maximum of 28.

(*c*) *Constriction*
The reduction in area after failure should be visible to the naked eye.

(*d*) *Unit elongation at maximum load*
The unit elongation at maximum load (ε_{uk}) shall be at least equal to 0.035.

2.3.4.5. Relaxation

Prestressing steels are divided into relaxation classes which refer to the relaxation at 1000 hours (ρ_{1000}) for initial stresses equal to 0.6, 0.7 and 0.8 times f_{ptk}.
Three relaxation classes are defined

- class 1: normal relaxation characteristics for wires and strands
- class 2: improved relaxation characteristics for wires and strands
- class 3: relaxation characteristic for bars.

For the relaxation test procedure, see document RILEM RPC 15.

For design purposes, the values according to Fig. 2.3.3 can be used.

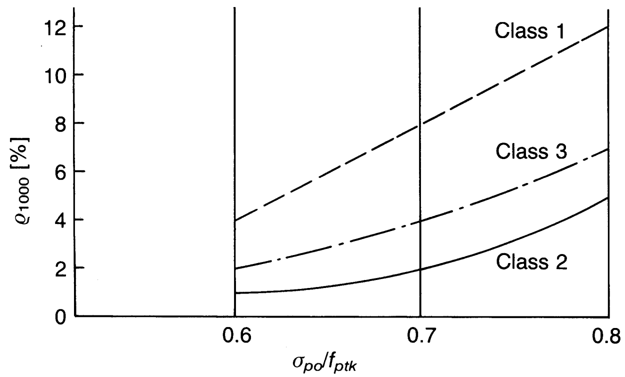


Fig. 2.3.3. Relaxation losses in % for different stress levels and relaxation classes

An indication of how relaxation varies with time up to 1000 hours is given in Table 2.3.1.

Table 2.3.1. Relationship between relaxation losses and time up to 1000 hours

Time in hours	1	5	20	100	200	500	1000
Relaxation losses as percentage of losses in 1000 hours	25	45	55	70	80	90	100

For an estimate of the relaxation up to 30 years the following formula may be applied

$$\rho_t = \rho_{1000} \left(\frac{t}{1000} \right)^k$$

where:

ρ_t is the relaxation after t hours
ρ_{100} is the relaxation after 100 hours
ρ_{1000} is the relaxation after 1000 hours

$k \approx \log(\rho_{1000}/\rho_{100})$

k to be 0.12 for relaxation class 1, and 0.19 for relaxation class 2.

Normally, the long-term values of the relaxation are taken from long-term tests. However, it may be assumed that the relaxation after 50 years and more is three times the relaxation after 1000 hours.

2.3.4.6. Fatigue behaviour

The characteristic fatigue strength (stress range) for 2×10^6 cycles, with a maximum stress being $0.7 f_{ptk}$, is given in Table 2.3.2

The values of Table 2.3.2 refer to the basic material as tested in the laboratory. For the performance of the embedded product used in the structure, refer to Table 6.7.2.

Table 2.3.2. Characteristic fatigue strength for 2×10^6 cycles

Basic material	$\Delta\sigma$ (N/mm^2)
Wires (cold drawn or hot rolled)	
• smooth	200
• indented	180
Strands	
• from smooth wires	190
• from indented wires	170
Smooth bars	200
Ribbed bars	180

2.3.5. Technological properties

2.3.5.1. Surface conditions

Prestressing steel should be free from defects, which may have occurred at any stage from manufacture up to installation, to a degree which could impair its performance as a prestressing element.

The surface of steel should be free from corrosion defects.

However, a thin film of rust which can be removed with a dry cloth may be tolerated.

2.3.5.2. Corrosion resistance

The time of exposure in ammonium thiocyanate till failure should not be less than the values given in the approval documents.

For the test method, see 'The FIP stress-corrosion test with ammonium thiocyanate', FIP Special Report 1988/1. The test is carried out with a tension of $0.8F_{ptk}$.

The recommended lowest values of exposure are given in Table 2.3.3.

Table 2.3.3. Stress-corrosion test with ammonium thiocyanate: recommended lowest values of exposure time

Product	Lowest exposure time to failure (hours)	Exposure time to failure of 50% of test samples (hours)
Wire	1.5	4
Strand	1.5	4
Bar < 12 mm	20	50
Bar 12–25 mm	60	250
Bar 25–40 mm	100	400

2.3.5.3. Thermal expansion

Within the temperature range from $-20°C$ to $100°C$ the coefficient of thermal expansion may be taken as $10 \times 10^{-6}/°C$.

3. GENERAL MODELS

This chapter contains engineering models describing the mechanical behaviour of reinforced concrete sub-elements. These models may serve either as a background for simpler models used in the operational parts of the Model Code, or as an input to more advanced design methods.

However, further assessment of their reliability is needed before these models can be used directly in practical design.

This chapter will be further developed in the future.

3.1. BOND STRESS–SLIP RELATIONSHIP

Under well defined conditions, it is possible to consider that there is an average 'local bond' versus 'local slip' relationship, statistically acceptable.

The bond stress–slip relationship depends on a considerable number of influencing factors like bar roughness (related rib area), concrete strength, position and orientation of the bar during casting, state of stress, boundary conditions and concrete cover.

Therefore the bond stress–slip curve, presented in Fig. 3.1.1 can be considered as a statistical mean curve, applicable as an average formulation for a broad range of cases. Further reliability handling would be needed to derive design bond stress–slip curves.

The first curved part refers to the stage in which the ribs penetrate into the mortar matrix, characterized by local crushing and micro-cracking. The horizontal level occurs only for confined concrete, referring to advanced crushing and shearing off of the concrete between the ribs. The decreasing branch refers to the reduction of bond resistance due to the occurrence of splitting cracks along the bars. The horizontal part represents a residual bond capacity, which is maintained by virtue of a minimum transverse reinforcement, keeping a certain degree of integrity intact.

With regard to the generation of bond stresses the following considerations apply.

Reinforcement and concrete have the same strain ($\varepsilon_s = \varepsilon_c$) in those areas of the structure which are under compression and in uncracked parts of the structure under tension.

In cracked cross-sections the tension forces in the crack are transferred by the reinforcing steel. In general, the absolute displacements of the steel u_s, and of the concrete u_c between two cracks or along the transmission length l_t are different.

Due to the relative displacement $s = u_s - u_c$ bond stresses are generated between the concrete and the reinforcing steel. The magnitude of these bond stresses depends predominantly on the surface of the reinforcing steel, the slip s, the concrete strength f_c and the position of the reinforcing steel

during concreting. Between cracks or along the transmission length l_t, a part of the tension force of the reinforcing steel, acting in the crack, is transferred into the concrete by bond (tension stiffening effect).

The local decrease of the relative displacement along the transmission length l_t is characterized by the strain difference

$$ds/dx = \varepsilon_s - \varepsilon_c$$

Depending on the selection of the coefficient α ($0 \leqslant \alpha \leqslant 1$) in eq. (3.1–1) all usual forms of a bond stress–slip relationship can be modelled, starting from a bond characteristic with a constant stress ($\alpha = 0$) up to a bond stress–slip relationship with linear increasing bond stress ($\alpha = 1$).

The parameters given in Table 3.1.1 are valid for ribbed reinforcing steel with a related rib area $A_{sr} \approx A_{sr,\min}$ according to relevant international standards, depending on the main influencing factors: confinement, bond condition and concrete strength.

Table 3.1.1. Parameters for defining the mean bond stress–slip relationship (according to eqs (3.1–1) to (3.1–4))

	Column 2	Column 3	Column 4	Column 5
	Unconfined concrete*		Confined concrete†	
	Good bond conditions	All other bond conditions	Good bond conditions	All other bond conditions
s_1	0.6 mm	0.6 mm	1.0 mm	1.0 mm
s_2	0.6 mm	0.6 mm	3.0 mm	3.0 mm
s_3	1.0 mm	2.5 mm	Clear rib spacing	Clear rib spacing
α	0.4	0.4	0.4	0.4
$\tau_{\max}$	$2.0\sqrt{f_{ck}}$	$1.0\sqrt{f_{ck}}$	$2.5\sqrt{f_{ck}}$	$1.25\sqrt{f_{ck}}$
τ_f	$0.15\tau_{\max}$	$0.15\tau_{\max}$	$0.40\tau_{\max}$	$0.40\tau_{\max}$

*Failure by splitting of the concrete.
†Failure by shearing of the concrete between the ribs.

3.1.1. Local bond stress–slip model

For monotonic loading the bond stresses between concrete and reinforcing bar can be calculated as a function of the relative displacement s according to eqs (3.1-1) to (3.1-4)

$$\tau = \tau_{\max}(s/s_1)^{\alpha} \text{ for } 0 \leqslant s \leqslant s_1 \quad (3.1\text{–}1)$$

$$\tau = \tau_{\max} \text{ for } s_1 < s \leqslant s_2 \quad (3.1\text{–}2)$$

$$\tau = \tau_{\max} - (\tau_{\max} - \tau_f)\left(\frac{s - s_2}{s_3 - s_2}\right) \text{ for } s_2 < s \leqslant s_3 \quad (3.1\text{–}3)$$

$$\tau = \tau_f \text{ for } s_3 < s \quad (3.1\text{–}4)$$

See also Fig. 3.1.1.

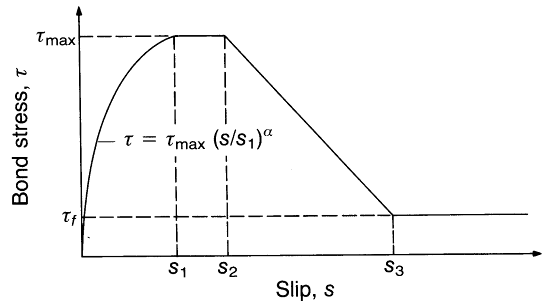


Fig. 3.1.1. Analytical bond stress–slip relationship (monotonic loading)

For bars with a related rib area $A_{sr} > A_{sr,\min}$ the bond strength $\tau_{\max}$ increases and the characteristic slip value s_1 decreases. This influence is, however, neglected. Furthermore the stiffness of the increasing branch of the bond stress–slip relationship differs from confined concrete to unconfined concrete, but this influence is also neglected. Finally the dependence of bond properties upon concrete compaction and curing is neglected as well.

The values in the Tables 3.1.1 and 3.1.2 are applicable only in loading states for which the concrete is not subjected to lateral tension.

The values given in Table 3.1.1, columns 2 and 3, are valid for a concrete cover $c = 1\phi_s$, and a minimum transverse reinforcement equal to

$$A_{st,\min} = 0.25nA_s$$

where

A_{st} area of stirrups (two legs) over a length equal to the anchorage length
n number of bars enclosed by stirrups
A_s area of one bar.

The values in columns 4 and 5 are valid for well confined concrete (concrete cover $c \geqslant 5\phi_s$, clear spacing $> 10\phi_s$ or closely spaced transverse (enclosing) reinforcement with an area $A_{st} > nA_s$) or high transverse pressure ($p \geqslant 7.5$ MPa as average transverse pressure under design load).

If the transverse reinforcement A_{st} is

$$A_{st,\min} < A_{st} < nA_s$$

or the transverse pressure p is

$$0 < p < 7.5\,\text{MPa}$$

the values for s_1, s_3, $\tau_{\max}$ and τ_f may be interpolated linearly between the values for unconfined and for confined concrete respectively. If a transverse reinforcement $A_{st} > A_{st,\min}$ is present simultaneously with a transverse pressure the effects may be added.

The values given in Table 3.1.1 are valid for those parts of the reinforcing bars which are a distance of $x > 5\phi_s$ from the crack.

For those parts of the reinforcing bar which are a distance $x \leqslant 5\phi_s$ from the next transverse crack, the bond stress τ and the slip s are to be reduced by the factor λ, where

$$\lambda = 0.2\,\frac{x}{\phi_s} \leqslant 1$$

The parameters given in Table 3.1.2 are valid for smooth reinforcing steel, depending on the main influencing factors: roughness of the bar surface, bond conditions and concrete strength. They are valid for confined and unconfined concrete.

Table 3.1.2. Parameters for defining the bond stress–slip relationship of smooth bars (according to eqs (3.1.–1) to (3.1–4))

	Cold drawn wire		Hot rolled bars	
	Good bond conditions	All other bond conditions	Good bond conditions	All other bond conditions
$s_1 = s_2 = s_3$	0.01 mm	0.01 mm	0.1 mm	0.1 mm
α	0.5	0.5	0.5	0.5
$\tau_{max} = \tau_f$	$0.1\sqrt{f_{ck}}$	$0.05\sqrt{f_{ck}}$	$0.3\sqrt{f_{ck}}$	$0.15\sqrt{f_{ck}}$

The parameters given in Tables 3.1.1 and 3.1.2 are mean values.

It has to be kept in mind, that the scatter of different test series is considerable especially for small values of the slip. For a given value of the slip the coefficient of variation of the bond stresses may amount to approx. 30%. The scatter is due to the use of different test specimens and the hereby created different states of stress in the concrete surrounding the reinforcing bar, to the different measuring techniques, and to the different loading and deformation velocities. The heterogeneity of the concrete and the geometry of the reinforcing bars (related rib area, diameters) have also a significant influence on the $\tau - s$ relationship. The designer should take account of this scatter as far as possible, at least in the cases where a more accurate design is necessary.

The unloading branch of the bond stress–slip relationship is linear and valid for the increasing and horizontal part of the diagram. The slope S (see Fig. 3.1.2) is independent of the slip value s, and has an average value of $S = 200\,N/mm^3$.

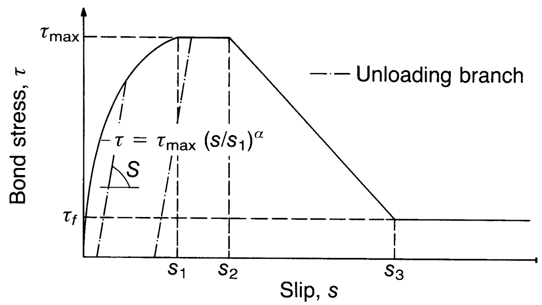


Fig. 3.1.2. Unloading branch of the τ – s relationship

3.1.2. Influence of creep

Creep influences will reduce the slope of the increasing part of the $\tau - s$ relationship.

The creep displacements can be described with isochrone curves (see Fig. 3.1.3). The slip $s_{n,t}$ due to a permanent load or a repeated loading can be calculated according to eq. (3.1–5)

$$s_{n,t} = s(1 + k_{n,t}) \qquad (3.1\text{–}5)$$

where the displacement factor k_t for a permanent load can be calculated according to eq. (3.1–6)

$$k_t = (1 + 10t)^{0.080} - 1 \qquad (3.1\text{–}6)$$

where t is the load duration (hours).

For a repeated loading the displacement factor k_n can be determined by eq. (3.1–7)

$$k_n = (1 + n)^{0.107} - 1 \qquad (3.1\text{–}7)$$

where n is the number of load cycles.

The validity of this relation is restricted to the ascending branch of the bond–slip relationship.

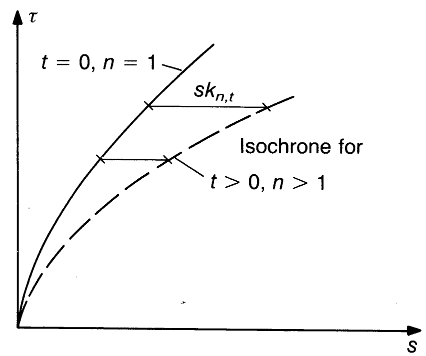


Fig. 3.1.3. Creep effects on the τ – s curve

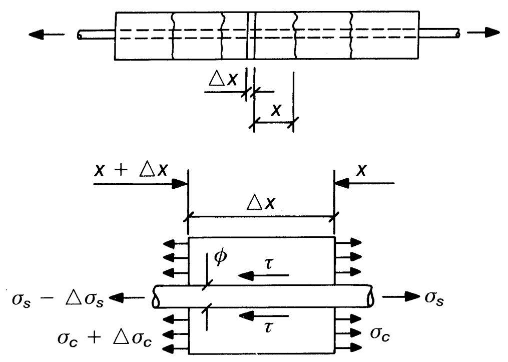


Fig. 3.1.4. Bond stress–slip model: $\varepsilon_{s,m}\Delta x = \varepsilon_{c,m}\Delta x + \Delta s$ (*slip*)

3.1.3. Applications of the model

As possible applications of the bond stress–slip model, the following cases may be mentioned.

(*a*) *Reinforced concrete tie*. Taking into account the equilibrium and compatibility conditions in an elementary length of a reinforced concrete tie, as well as its boundary conditions, it is possible to use the model described in the previous clauses to predict crack formations and the elongation of the tie. Tension stiffening effects may thus be studied.

(*b*) *Anchorage of bars*. Similarly, and under different boundary conditions, the steel–stress versus the pull-out displacement of a bar may be studied.

3.2. TENSION STIFFENING EFFECTS

3.2.1. Definition

Section 3.2 gives simplified constitutive laws for reinforced or prestressed concrete in pure tension after cracking. From the first crack up to yielding distinction should be made between the crack formation phase, in which new cracks occur, and the stabilized cracking phase in which only crack widening is supposed to occur.

If the tension stiffening effect is neglected, the stiffness of a reinforced concrete bar or a structural member is underestimated.

In a cracked cross-section all tensile forces are balanced by the steel only. However, between adjacent cracks, tensile forces are transmitted from the steel to the surrounding concrete by bond forces. The contribution of the concrete may be considered to increase the stiffness of the tensile reinforcement. Therefore this effect is called the 'Tension Stiffening Effect'.

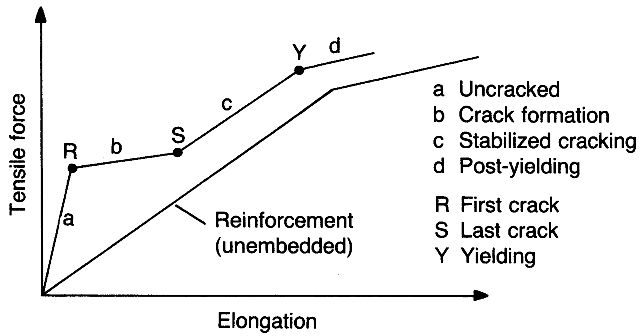


Fig. 3.2.1. Idealized behaviour of a reinforced concrete tie

The distinction between the stages uncracked concrete, crack formation phase, stabilized cracking and post-yielding is helpful in estimating deformation, crack width and damping.

More detailed information on the cracking process is given in clause 7.4.3.1.

For plain concrete under tension, see clause 2.1.4.4.2.

First crack

When the first crack occurs the distribution of steel and concrete strain within the transmission length l_t is given in Fig. 3.2.2.

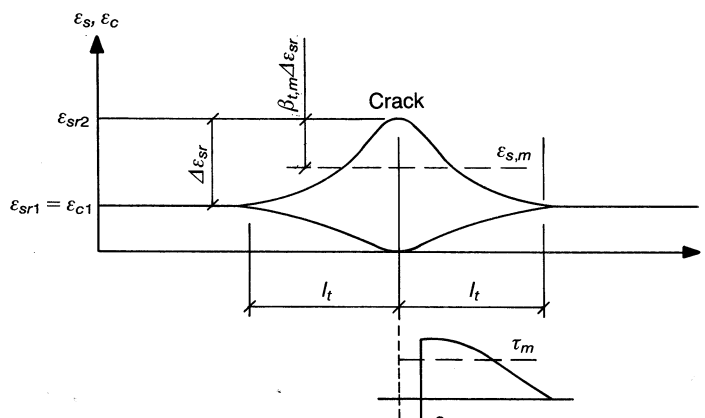


Fig. 3.2.2. Single crack

3.2.2. Crack pattern

During the state of crack formation one crack after the other occurs thus decreasing the stiffness of the member. When cracks appear, single cracks play an important role. In this state some part of the area between the cracks remains in state I ($\varepsilon_s = \varepsilon_c$). When the crack pattern has stabilized, the distance between the cracks s_r is equal to or less than twice the transmission length l_t (length over which slip between steel and concrete occurs).

The mean steel strain may be expressed as

$$\varepsilon_{s,m} = \varepsilon_{s2} - \beta_{t,m}\Delta\varepsilon_{sr} = \varepsilon_{s2} - \beta_{t,m}(\varepsilon_{sr2} - \varepsilon_{sr1}) \qquad (3.2\text{–}1)$$

where $\beta_{t,m}$ is considered to be an integration factor for the steel strain along the transmission length. (In clause 7.4.3.1 the abbreviation $\beta = \beta_{t,m}$ is used.)

$\beta_{t,m} = 0.60$ for pure tension

$\varepsilon_{s,m}$ is the mean steel strain
ε_{s1} is the strain of reinforcement in uncracked concrete
ε_{s2} is the strain of reinforcement in the crack
ε_{sr1} is the steel strain at the point of zero slip under cracking forces reaching $f_{ctm}(t)$
ε_{sr2} is the strain of reinforcement at the crack under cracking forces reaching $f_{ctm}(t)$; if the internal forces are lower than or equal to the cracking forces (e.g. in a working joint), then $\varepsilon_{sr2} = \varepsilon_{s2}$
$\Delta\varepsilon_{sr}$ is the increase of steel strain in the cracking state.

The bond force F_b transmitted along the transmission length can be described by

$$F_b = \phi\pi\tau_m l_t = A_s E_s \Delta\varepsilon_{sr} \qquad (3.2\text{–}2)$$

where

ϕ is the bar diameter
τ_m is the mean value of the bond strength (see also subsection 7.4.3).

After crack formation
After the crack formation has finished, the mean spacing between cracks $s_{r,m}$ can be taken as

$$s_{r,m} = \tfrac{2}{3}2l_t = \tfrac{4}{3}l_t \qquad (3.2\text{–}3)$$

Then the transferred bond force is reduced according to the reduced transmission length $l_{t,m} = \frac{2}{3}l_t$

$$F_{b,m} = \tfrac{2}{3}F_b = \tfrac{2}{3}A_s E_s \Delta\varepsilon_{sr} \qquad (3.2\text{–}4)$$

Accordingly the reduction of the steel strain from the crack to the point in the middle between the cracks is given by

$$\Delta\varepsilon_{s,m} = \tfrac{2}{3}\Delta\varepsilon_{sr} \qquad (3.2\text{–}5)$$

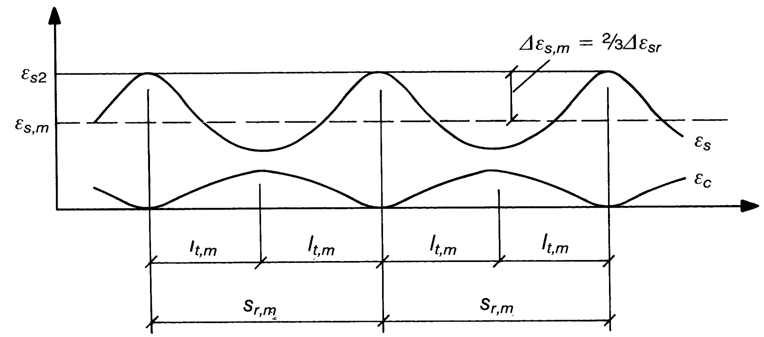


Fig. 3.2.3. Stabilized crack pattern with $s_{r,m} = \frac{4}{3} l_t$

The mean strain over the total member may be taken as

$$\varepsilon_{s,m} = \varepsilon_{s2} - \beta_{t,m}\Delta\varepsilon_{s,m} = \varepsilon_{s2} - \beta_{t,m}\tfrac{2}{3}\Delta\varepsilon_{sr} = \varepsilon_{s2} - \beta_t\Delta\varepsilon_{sr} \qquad (3.2\text{–}6)$$

where

$\beta_t = \frac{2}{3}\beta_{t,m} =$ 0.40 for instantaneous loading, and 0.25 for long-term and repeated loading.

For calculation of transmission length l_t, see clause 7.4.3.1.

3.2.3. Stress–strain relation of steel embedded in concrete

In a cracked zone the strain of the reinforcement varies along the bar (Fig. 3.2.2 and 3.2.3). The overall deformability of the reinforcement may be described by the mean value of the steel strain. When calculating the tension stiffening effect in this way, distinction should be made between the crack formation phase and the stabilized crack pattern.

For normal cases the steel stress at the last crack may be taken as

$$\sigma_{srn} = 1.3\sigma_{sr} \qquad (3.2\text{–}7)$$

with $\sigma_{sr} = \sigma_{sr1}$ the steel stress in the first crack.

In this case formula (3.2–9) reads

$$\varepsilon_{sm} = \varepsilon_{s2} - \frac{\beta_t(\sigma_s - \sigma_{sr}) + (1.3\sigma_{sr} - \sigma_s)}{0.3\sigma_{sr}}(\varepsilon_{sr2} - \varepsilon_{sr1}) \qquad (3.2\text{–}10)$$

For a member under pure tension σ_{sr} can be calculated as

$$\sigma_{sr} = N_r/A_s$$

For short-term loading

$$N_r = A_c(1 + \alpha\rho)f_{ct}$$

where

α is the modular ratio, and ρ is the geometrical ratio of reinforcement.

For long-term loading creep and shrinkage should be taken into consideration.

For different cases of application (minimum reinforcement, deflection, stability) different fractile values of the tensile strength f_{ct} should be applied.

It is proposed to take

- for deflections, the mean or the lower fractile value of f_{ct}
- for minimum reinforcement, an upper fractile
- for stability verifications, the mean value
- for crack width calculation, also the mean value.

It is assumed that in practice only deformed bars are used.

For practical application the tension stiffening effect may be taken into account by a modified stress–strain relation of the embedded reinforcement ($\sigma_s - \varepsilon_{s,m}$ relation) as follows:

(a) uncracked

$$\varepsilon_{s,m} = \varepsilon_{s1} \qquad 0 < \sigma_s \leqslant \sigma_{sr1} \qquad (3.2\text{–}8)$$

(b) crack formation phase

$$\varepsilon_{s,m} = \varepsilon_{s2} - \frac{\beta_t(\sigma_s - \sigma_{sr1}) + (\sigma_{srn} - \sigma_s)}{\sigma_{srn} - \sigma_{sr1}}(\varepsilon_{sr2} - \varepsilon_{sr1})$$

$$\sigma_{sr1} < \sigma_s \leqslant \sigma_{srn} \qquad (3.2\text{–}9)$$

(c) stabilized cracking

$$\varepsilon_{s,m} = \varepsilon_{s2} - \beta_t(\varepsilon_{sr2} - \varepsilon_{sr1}) \qquad \sigma_{srn} < \sigma_s \leqslant f_{yk} \qquad (3.2\text{–}11)$$

(d) post-yielding

$$\varepsilon_{s,m} = \varepsilon_{sy} - \beta_t(\varepsilon_{sr2} - \varepsilon_{sr1}) + \delta\left(1 - \frac{\sigma_{sr1}}{f_{yk}}\right)(\varepsilon_{s2} - \varepsilon_{sy})$$

$$f_{yk} < \sigma_s < f_{tk} \qquad (3.2\text{–}12)$$

where

ε_{sy} is the strain at yield strength
σ_s is the steel stress in the crack
σ_{sr1} is the steel stress in the crack, when first crack has formed
σ_{srn} is the steel stress in the crack, when stabilized crack pattern has formed (last crack)
$\beta_t = 0.40$ for short-term loading (pure tension)
$\beta_t = 0.25$ for long-term or repeated loading (pure tension)
$\delta = 0.8$; coefficient to take into account the ratio f_{tk}/f_{yk} and the yield stress f_{yk}.

The value $\delta = 0.8$ is valid for ductile steel (type A) and $f_{yk} = 500$ MPa.

When calculating the effects of imposed deformations, the inclined line according to eq. (3.2–9) may be replaced by the dotted line shown in Fig. 3.2.4.

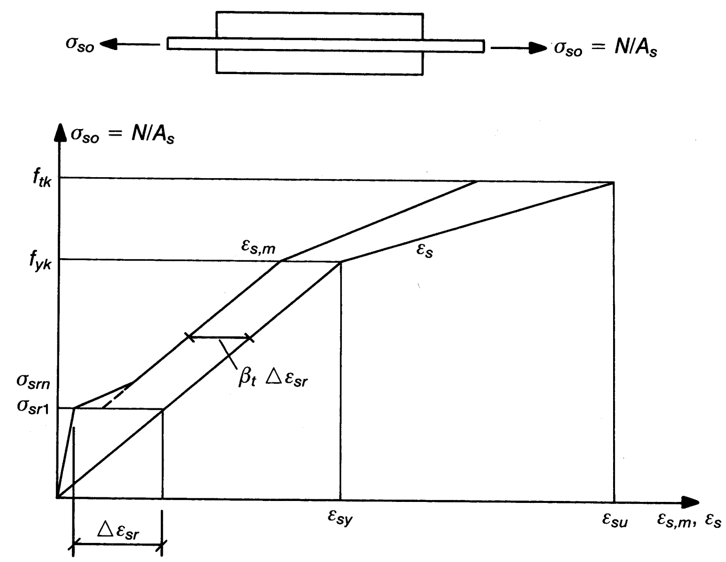


Fig. 3.2.4. Simplified stress–strain relationship of embedded reinforcing steel

3.3. LOCAL COMPRESSION

The compressive bearing capacity f_{cc}^* of locally loaded concrete is governed by the failure mechanisms described in the following. Corresponding approximate models and limiting f_{cc}^*-values presented in this chapter may be used when other more precise models are not available.

The lowest of the f_{cc}^*-values corresponding to the failure modes of sub-sections 3.3.1, 3.3.2 and 3.3.3, will be the f_{cc}^*-value valid in each case.

Dispersion of concentrated forces causes biaxial or triaxial compression immediately under the load, whereas it produces transverse tension further away.

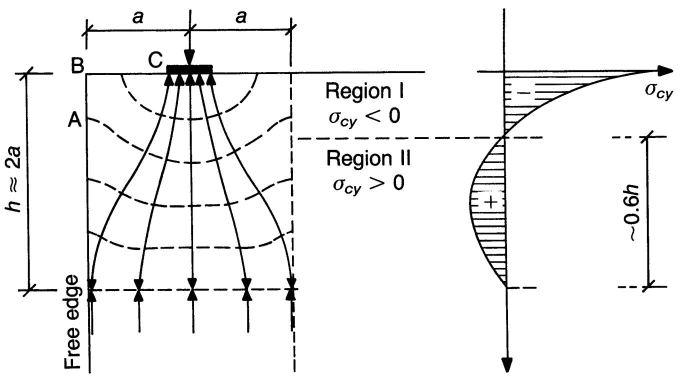


Fig. 3.3.1. Stress field under a concentrated load

Failure may be observed in the upper region (I) by transverse tension (splitting) or by compression (crushing). Similarly, a tension failure may occur in the lower region (II) by transverse tension (bursting).

Local spalling of corners (e.g. ABC in Fig. 3.3.1) is not considered here.

Reliability aspects, not included in this modelling, should appropriately be handled, taking into account the increased model uncertainties.

3.3.1. Spalling near the end face of a partially loaded surface

Near the end face of a partially loaded surface, lateral dilatancy of the locally compressed concrete is hindered by the surrounding mass of non-loaded concrete.

This surrounding concrete is therefore subjected to an expansion, possibly leading to transverse cracking. In order to avoid such a cracking, the local compression f_{cc}^* should be limited according to eq. (3.3–1):

$$f_{cc}^* \;=\; f_{cc}\sqrt{(A_2/A_1)} \not> 4f_{cc} \qquad (3.3\text{–}1)$$

where

- f_{cc}^* is the bearing capacity of concrete under local compression
- f_{cc} is the compressive strength of concrete under uniaxial stress (reductions of this strength in the sense of clause 6.2.2.2 are also applicable)
- A_1 is the loaded area
- A_2 is the cross-section of the surrounding concrete into which the stress field is developed (leading to a final uniform longitudinal stress distribution).

For helically reinforced concrete, or in presence of closed stirrups, the relevant provisions of confined concrete in section 3.5 have to be followed (leading to an additional increase of strength), under the condition that the transverse reinforcement is arranged in the first third of the stress field length close to the load.

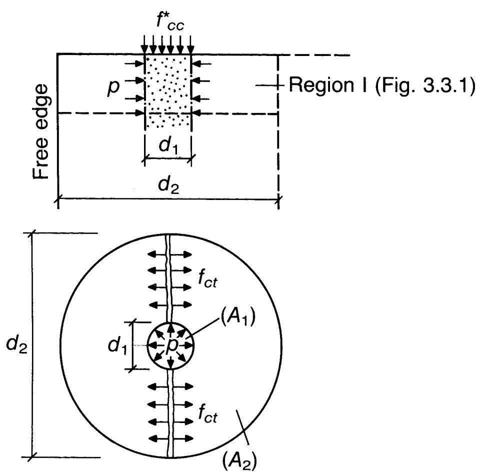


Fig. 3.3.2. Transverse expansion of concrete near a partially compressed end-face, including transverse cracking (spalling)

The following equations apply to Fig. 3.3.2:

equilibrium before spalling

$$pd_1 \;=\; f_{ct}(d_2 - d_1) \quad d_2 > d_1$$

$$p \;=\; \frac{f_{cc}}{10}\,\frac{d_2 - d_1}{d_1}$$

triaxial effect

$$f_{cc}^* \;=\; f_{cc} + 5p \;=\; f_{cc} + 0.5f_{cc}(d_2 - d_1)/d_1$$

with

$$d_2 \approx 2 \text{ to } 4d_1$$

$$f_{cc}^* \approx 0.7f_{cc}\sqrt{(A_2/A_1)}$$

Because of favourable size effects, however, the basic concrete strength may be taken as $1.3f_{cc}$. Thus

$$f_{cc}^* \approx f_{cc}\sqrt{(A_2/A_1)}$$

The neglect of an eventual frictional component at the edge of region I is compensated by an overestimation of the tensile resistance.

The explanation presented above is approximate. The overestimation of f_{ct}, acting over the full length of the crack, is compensated by the neglect of the shear stress on the lower surface of region I.

A refinement could be obtained by adding a compatibility equation.

As an 'effective' surrounding area A_2, the minimum area inscribed in the actual total one can be taken, geometrically similar to the loaded area A_1 and having the same centre.

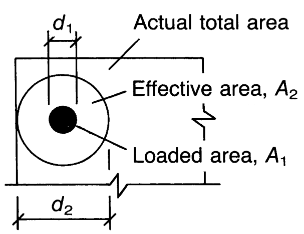


Fig. 3.3.3. Effective surrounding area A_2

For a rectangular loaded area bh (with $h < 2b$) an equivalent circular area could be used in the above modelling, with $d_{1,eq} = 1.125\sqrt{A_1}$.

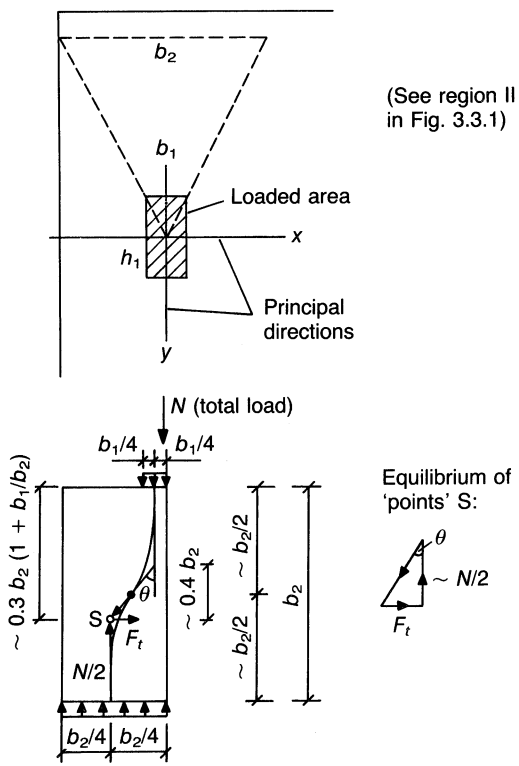


Fig. 3.3.4. Simplified model for the estimation of the bursting forces across each direction

In the simplified model shown in Fig. 3.3.4, the force trajectory is approximately considered as consisting of two circular arcs. Equilibrium of 'points' S gives

$$F_t = (N/2)\tan\theta = \frac{(N/2)(b_2/4 - b_1/4)}{0.4b_2}$$

$$F_t = 0.3N(1 - b_1/b_2)$$

3.3.2. Splitting in deeper zones

Transverse tensile bursting forces in the more remote region of the stress field should be estimated for both principal directions of the loaded area and should be resisted by the tensile strength of the concrete itself or by specially provided reinforcement (see also subsections 6.9.12 and 9.1.1) if the concrete is expected to crack longitudinally.

Substituting the stress field by two force-trajectories, within a length equal to the width of the effective surrounding concrete area, the bursting forces may be estimated as

$$F_{t,x} = 0.3N(1 - b_1/b_2) \qquad (3.3\text{–}2)$$

$$F_{t,y} = 0.3N(1 - h_1/h_2) \qquad (3.3\text{–}3)$$

where

$$N = b_1 h_1 f_{cc}^*$$

These forces are resisted as follows

$$F_{t,x} < f_{ct} 0.6 b_2 h_1 \quad \text{or} \quad A_{sx} f_{yk} \qquad (3.3\text{–}4)$$

$$F_{t,y} < f_{ct} 0.6 h_2 b_1 \quad \text{or} \quad A_{sy} f_{yk} \qquad (3.3\text{–}5)$$

where

$0.6b_2$ and $0.6h_2$ are approximate values of the respective heights of the tension region of the stress field

A_{sx} and A_{sy} are the corresponding cross-sections of well anchored reinforcement transversely arranged within the tension region.

This is a conservative approach considering the spreading of the force N separately in each direction x, y. In fact, for loaded areas approaching the square form, a biaxial spreading of the force N may mobilize substantially lower transverse tensile stresses, leading to considerably higher bearing capacities f_{cc}^*.

Care about longitudinal crack control, however, as well as uncertainties about unforeseen eccentricities, justify the use of the 'plane stress approach' in current design cases.

A slightly less conservative value may be found if the internal lever arm of bursting forces is taken equal to $0.5b_1$. In such a case, the numerical factor 0.30 in eq. (3.3–2) is reduced to 0.25 (see eq. (6.9–1)).

Equating of eqs (3.3–2) to (3.3–5) leads to an expression of the bearing capacity of locally loaded concrete, corresponding to the failure mode under consideration, see eq. (3.3–6)

$$0.3 f_{cc}^* b_1 h_1 \left(1 - \frac{b_1}{b_2}\right) = f_{ct} 0.6 b_2 h_1 \text{ or } A_{sx} f_{yk} \qquad (3.3\text{–}6)$$

Defining the mechanical volumetric percentage of bursting reinforcement in the direction x as

$$\omega_x = \frac{A_{sx}}{h_1 0.6 b_2} \frac{f_{yk}}{f_{cc}} \qquad (3.3\text{–}7)$$

it follows that

$$0.3 f_{cc}^* b_1 \left(1 - \frac{b_1}{b_2}\right) = 0.6 b_2 \left(\omega_x \text{ or } \frac{f_{ct}}{f_{cc}}\right) \text{ or}$$

$$\frac{f_{cc}^*}{f_{cc}} = 2 \frac{(b_2/b_1)^2}{(b_2/b_1) - 1} \left(\omega_x \text{ or } \frac{f_{ct}}{f_{cc}}\right) \qquad (3.3\text{–}8)$$

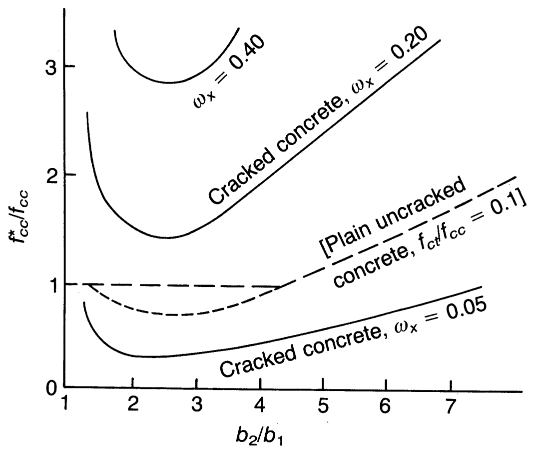


Fig. 3.3.5. Bursting resistance in relation to ω_x *and* b_2/b_1

The triaxial compressive stresses under the loaded area, in case of a relatively small loading area or significant confinement can be that large local pulverization of the concrete occurs. This pulverization occurs until no further volume reduction is possible. The pulverized material causes a quasi-hydrostatic pressure on the confining concrete, which may lead to local wedging off of a part of the surface area.

3.3.3. Surface crushing

When a relatively small area of a very large surface is compressed, or when significant confining capacity is available, a type of local bearing failure may occur, comparable to Prandtl's wedge.

If not more precisely calculated, the average bearing capacity can be calculated with the expression

$$f^*_{cc}/f_{cc} = 12.5\sqrt{(40/f_{cc})} \qquad (3.3\text{–}9)$$

(f_{cc} in MPa).

However, if limited penetration is to be considered, f^*_{cc}-values should not be taken higher than $4f_{cc}$.

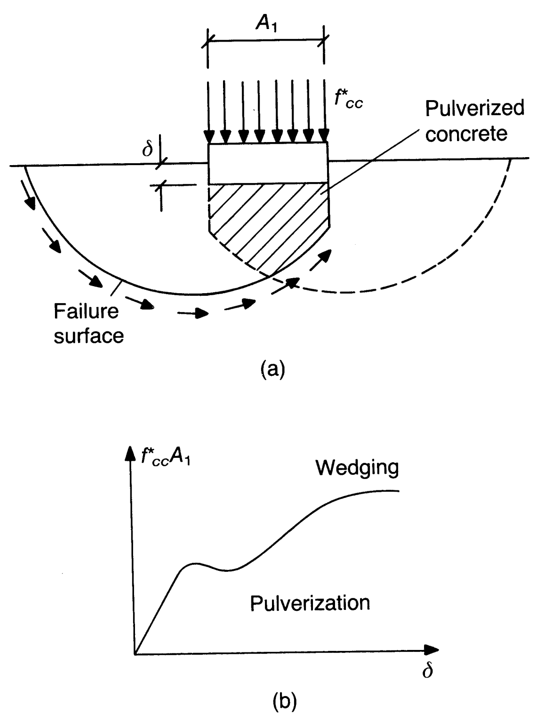


Fig. 3.3.6. Surface crushing (Prandtl's wedge)

In the compression–tension quadrant the biaxial failure envelope of plain concrete gives a compressive stress at failure, σ_{c2}, smaller than the uniaxial compressive strength f_{cm}. For the combination $\sigma_{c1} > 0$, $\sigma_{c2} < 0$, $\sigma_{c3} = 0$ the multiaxial failure criterion in clause 2.1.3.4 gives the following failure envelope (compare to eqs 2.1–8 through 2.1–11)

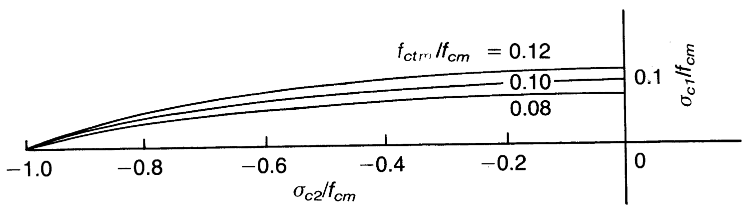


Fig. 3.4.1. Multiaxial failure envelope

3.4. BIAXIAL COMPRESSION AND TENSION

In reinforced concrete subjected to biaxial compression and tension, the concrete strength in the direction of the compressive stress, σ_{c2}, is reduced after cracking. This strength reduction is mainly because of the tensile stress, σ_{c1}, developed in the concrete between the cracks, due to tensile forces transferred by bond from the steel bars. Moreover, the concrete strips between the cracks are slender, and therefore less resistant to compression.

There is experimental evidence that the reduction in compressive strength increases as the crack spacing decreases. Therefore, any effect that decreases crack spacing (e.g. the use of smaller diameter bars) is expected to increase this strength reduction. For this reason, and also because of the higher tension forces they transfer to the concrete between the cracks, high bond deformed bars are expected to reduce the concrete strength in the direction of the compressive stresses more than plain bars.

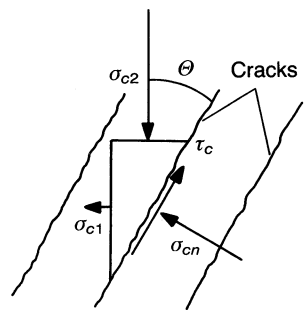


Fig. 3.4.2. Stresses acting in a crack

If the concrete cracks are at an angle θ to the direction of σ_{c2} (see Fig. 3.4.2), the normal and shear stress component on the plane of the crack are

$$\sigma_{cn} \approx |\sigma_{c2}| \sin^2 \theta \tag{3.4–1}$$

$$\tau_c \approx \tfrac{1}{2} |\sigma_{c2}| \sin 2\theta \tag{3.4–2}$$

The magnitude of the shear stress τ_c that can be developed in the crack is limited by the sum of

(*a*) the shear resistance of the interface due to concrete-to-concrete friction (given in clause 3.9.2.1 in terms of the normal stress component $\sigma_c = \sigma_{cn}$), and

(*b*) the maximum shear forces that can be transferred by dowel action by the reinforcing bars crossing a unit area of the crack (given in section 3.10 in terms of the axial stress σ_s in the steel bars, and other parameters, and becoming zero after yielding of bars).

If the cracks are not parallel to the direction of the compressive stresses, the latter will have to be transferred across the cracks by a combination of concrete-to-concrete friction (interface shear and compression normal to crack, section 3.9) and dowel action (section 3.10). This results in a further reduction of the concrete compressive strength, which is greatest when the cracks are at 45° to the direction of the applied compressive stresses, and smallest when they are parallel to it.

So, a limit is also implicitly imposed on the value of σ_{c2} that can be developed in the concrete. This limit value, which can be taken as a reduced compressive strength of concrete in the presence of cracks at an angle θ to the applied compression, is a maximum when the cracks are parallel to the applied compressive stress ($\theta = 0$), and a minimum when they are at an angle $\theta = 45°$ to it.

For cracking parallel to the direction of applied compression, the reduced design concrete strength due to transverse tension can be taken as

$$f_{cd}^{*} = f_{cd}/(1 + k\varepsilon_1/\varepsilon_{c0}) \tag{3.4–3}$$

in which ε_1 is the average (smeared) tensile strain of cracked reinforced concrete normal to the direction of applied compression, and k a coefficient which depends on the surface roughness and the diameter of the bars. For medium diameter deformed bars, k can be taken equal to 0.1, whereas for small diameter smooth welded wire mesh, k is approximately equal to 0.2.

The reduction in concrete compressive strength due to simultaneously acting transverse tension depends on the magnitude of the average (smeared) tensile strain in the transverse direction, on the inclination of the cracks with respect to the direction of the compressive stress, on the surface roughness and diameter of the steel bars, etc.

3.5. DATA FOR CONFINED CONCRETE

3.5.1. General

General mechanical characteristics of concrete confined by means of adequate closed stirrups or cross-ties are taken as those of unconfined concrete, except where given below.

Confinement results in a modification of the constitutive law of the concrete; higher strength and higher critical strains are achieved.

Nevertheless, most of the other basic mechanical characteristics are practically unaffected, at least as far as design is concerned.

Therefore, due to lack of theoretical and experimental data, moduli of elasticity (E, G), Poisson's ratio and coefficient of thermal expansion of confined concrete should be considered equal to those for unconfined concrete.

Long-term behaviour characteristics of confined concrete (shrinkage and creep) should be taken as those of unconfined concrete.

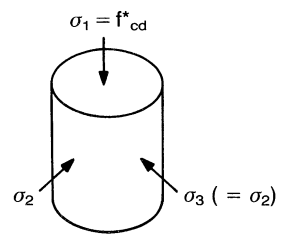


Fig. 3.5.1. Axially compressed concrete with lateral confinement

3.5.2. ULS under axial load-effects

3.5.2.1. Practical modelling

When a more precise study is not made, the following practical model may be used

(*a*) When axially compressed concrete reaches its plastic condition, confining-steel (closed stirrups or hoops) develops stresses close to its yield limit. Thus, the average confining stress laterally acting on concrete may be approximated by

$$\frac{\sigma_2}{f_{cd}} \approx \frac{\sigma_3}{f_{cd}} = \tfrac{1}{2}\omega_w \qquad (3.5\text{–}1)$$

where ω_w defines the volumetric mechanical ratio of confining steel.

(*b*) Taking into account the non-uniformity of distribution of these confining stresses, the 'effective lateral stress' may be approximated by the expression

$$\frac{\sigma_2}{f_{cd}} \approx \frac{\sigma_3}{f_{cd}} = \tfrac{1}{2}\alpha_n \alpha_s \omega_w \qquad (3.5\text{–}2)$$

where

α_n is a reduction factor expressing the effective concrete area in plan (depending on the hoop-pattern in the cross-section)
α_s is a reduction factor expressing the effective concrete area in elevation (depending on the spacing of hoops).

Examples

In Fig. 3.5.2(a):

$$\sigma = \frac{2A_s f_{yd}}{bs}$$

where s = spacing between hoops

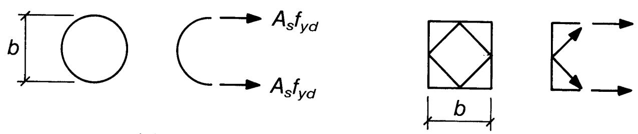


Fig. 3.5.2. Examples of confining reinforcement

$$\omega_w = \frac{4\pi b A_s}{\pi b^2}\frac{f_{yd}}{f_{cd}}$$

$$\frac{\sigma}{f_{cd}} = 0.5\omega_w$$

In Fig. 3.5.2(b):

$$\sigma = \frac{\left(2 + 2\frac{\sqrt{2}}{2}\right) A_s f_{yd}}{bs} = \frac{3.415 A_s f_{yd}}{bs}$$

$$\omega_w = \frac{\left(4 + 4\frac{\sqrt{2}}{2}\right) bA_s}{b^2 s}\frac{f_{yd}}{f_{cd}} = \frac{6.83 bA_s}{bs}\frac{f_{yd}}{f_{cd}}$$

$$\frac{\sigma}{f_{cd}} = 0.5\omega_w$$

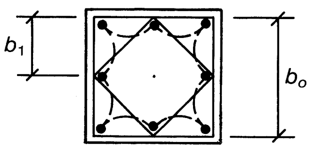


Fig. 3.5.3. Horizontal cross-section of column with non-uniform distribution of confining stresses

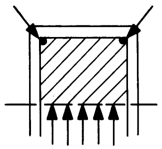


Fig. 3.5.4. Cross-section of beam with confining action of hoops

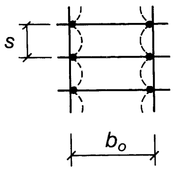


Fig. 3.5.5. Vertical cross-section of column with non-uniform distribution of confining stresses (in elevation)

Referring to Fig. 3.5.3, the coefficient α_n can be calculated as follows

$$\alpha_n \approx 1 - \frac{n(b_1^2/6)}{b_0^2} = 1 - \frac{8}{3}\frac{1}{n} \qquad (b_i < 200\,\text{mm})$$

where n is the total number of tied longitudinal bars. In the case of the compressive zone of beams, the neutral axis may be considered as a 'solid' border, hindering lateral expansion (Fig. 3.5.4).

The coefficient α_s follows from Fig. 3.5.5:

$$\alpha_s = \left(1 + \frac{1}{2}\frac{s}{b_0}\right) \qquad s < b_0/2$$

When a more precise analysis is not carried out, the values of the reduction factors α_n and α_s, may be taken from Fig. 3.5.6

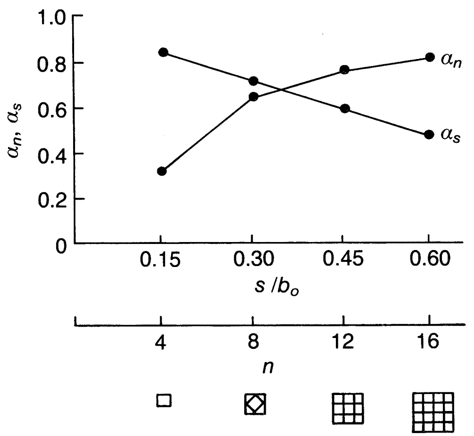


Fig. 3.5.6. Approximate values of α_n *and* α_s

Note. For circular columns and circular hoops

$$\alpha_n = 1 \qquad \alpha_s = \left(1 - \frac{1}{2}\frac{s}{b_0}\right)^2$$

For circular columns and spiral reinforcement

$$\alpha_n = 1 \qquad \alpha_s = \left(1 - \frac{1}{2}\frac{s}{b_0}\right)$$

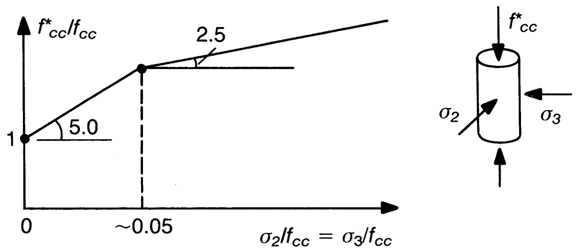


Fig. 3.5.7. Linearized approximation of compressive strength under triaxial axisymmetric loading

(*c*) When a more precise relationship between the triaxial compressive strength f^*_{cc} and the unconfined strength f_{cc} is not used, a linearized approximation may be adopted

$$f^*_{cc} = f_{cc}(1.000 + 2.50\alpha\omega_w) \quad \text{for } \sigma_2/f_{cc} < 0.05 \qquad (3.5\text{–}3)$$

or

$$f^*_{cc} = f_{cc}(1.125 + 1.25\alpha\omega_w) \quad \text{for } \sigma_2/f_{cc} > 0.05 \qquad (3.5\text{–}4)$$

where $\alpha = \alpha_n \alpha_s$.

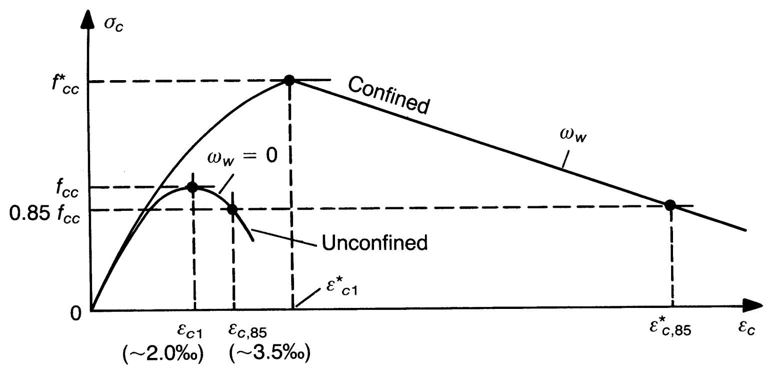


Fig. 3.5.8. Approximation for stress–strain relationship under triaxial axisymmetric conditions

Increased characteristic strength and strains given in this section are rather conservative; therefore there is no need for a model uncertainty factor (γ_{Rd}) or for an increased partial safety factor for the material.

Thus, $\gamma_{c,cf} = \gamma_c = 1.50$.

The coefficient 0.85 on f_{cd} in Fig. 3.5.9 takes account of the unfavourable effect of long-term loads.

(*d*) If a more precise model is not used, the following approximation may be applied in predicting the stress–strain relationship under triaxial axisymmetric conditions:

$$\varepsilon^*_{c1} = \varepsilon_{c1} + (f^*_{cc}/f_{cc})^2 \qquad (3.5\text{–}5)$$

$$\varepsilon^*_{c,85} = \varepsilon_{c,85} + 0.1\alpha\omega_w \qquad (3.5\text{–}6)$$

where $\alpha = \alpha_n \alpha_s$.

3.5.2.2. Idealized (σ — ε) diagram

(*a*) *Parabola-rectangle diagram*

For calculating the resistant load effects, if other more refined models are not available, a design diagram is used. See Fig. 3.5.9.

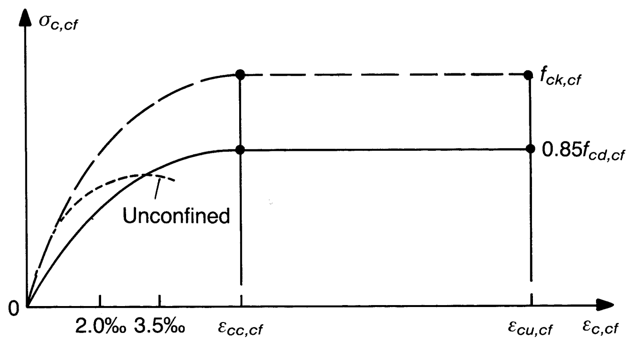
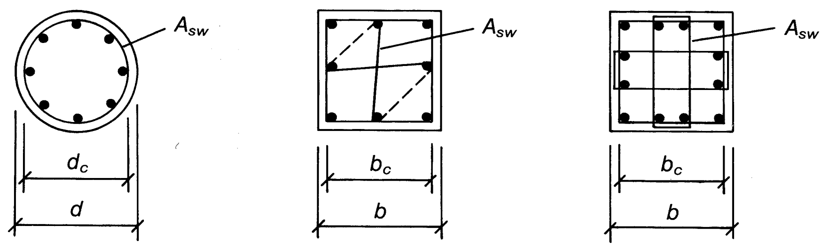


Fig. 3.5.9. Design (σ — ε) diagram for confined concrete

For the higher concrete grades, when a significant reduction of ε_{c1}- and $\varepsilon_{cu,85}$-values are applicable, correspondingly reduced values will be used in eqs (3.5–8) and (3.5–9).



$$\omega_{wd} = \frac{4A_{sw}}{d_c s} \cdot \frac{f_{yd}}{f_{cd}} \qquad \omega_{wd} = \frac{6A_{sw}}{b_c s} \cdot \frac{f_{yd}}{f_{cd}} \qquad \omega_{wd} = \frac{9A_{sw}}{b_c s} \cdot \frac{f_{yd}}{f_{cd}}$$

Fig. 3.5.10. Calculation of ω_{wd}

Appropriate triaxial models should be used. If more precise data are not available, the increased characteristic strength and strains may be estimated by the following equations

$$\left.\begin{aligned} f_{ck,cf} &= f_{ck}(1.000 + 5.0\sigma_2/f_{ck}) \quad \text{for } \sigma_2 < 0.05 f_{ck} \\ f_{ck,cf} &= f_{ck}(1.125 + 2.50\sigma_2/f_{ck}) \quad \text{for } \sigma_2 > 0.05 f_{ck} \end{aligned}\right\} \tag{3.5–7}$$

$$\varepsilon_{cc,cf} = 2.0 \times 10^{-3}(f_{ck,cf}/f_{ck})^2 \tag{3.5–8}$$

$$\varepsilon_{cu,cf} = 3.5 \times 10^{-3} + 0.2\sigma_2/f_{ck} \tag{3.5–9}$$

where $\sigma_2(=\sigma_3)$ is the effective lateral compression stress at ULS due to confinement.

Simplified models for the evaluation of σ_2 may be used, i.e.

$$\sigma_2/f_{ck} = 0.5\alpha\omega_{wd}$$

where

α is the effectiveness of confinement $= \alpha_n \alpha_s$, see eq. (3.5–2); α_n depends on the arrangement of stirrups in the cross-section, and α_s depends on the spacing of the stirrups
ω_{wd} is the design mechanical volumetric ratio of confining reinforcement.

$$\omega_{wd} = \frac{W_{s,trans}}{W_{c,cf}} \frac{f_{yd,trans}}{f_{cd}}$$

where

$W_{s,trans}$ is the volume of closed stirrups or cross-ties
$W_{c,cf}$ is the volume of confined concrete
$f_{yd,trans}$ is the design yield stress of transverse reinforcement
f_{cd} is the design strength of unconfined concrete.

(*b*) *Rectangular diagram*
If a confined concrete section is not entirely in compression, a simplified rectangular distribution of the compressive stresses can be taken as for unconfined concrete.

The mean steel strain $\varepsilon_{s,m}$ may be assessed on the basis of subsection 3.2.3 (tension stiffening effect).

In connection to this the following points should be observed

(*a*) An effective concrete area, subjected to tension, around the tensile reinforcement should be assessed.
(*b*) The mean concrete strain at the top of the cross-section $\varepsilon_{c,m}$ should be estimated taking into account the variation of the depth of the compression zone between adjacent cracks.
(*c*) An appropriate lever arm should be used.

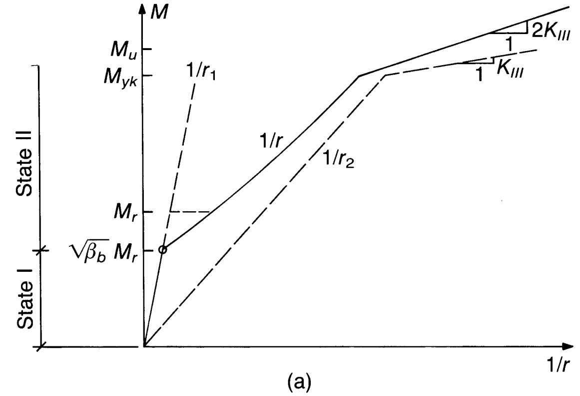
The solid line in Figs 3.6.1(a) and (b) represents the case which is most representative for actual practice; it includes a general reduction factor β_b for the concrete tensile strength, such as caused by the effects of shrinkage, sustained loading etc. If the concrete is in the virgin state and the loading is of short-term character, the dotted line is nearer to reality.

3.6. MOMENT–CURVATURE RELATIONSHIP

The mean curvature at any section of an element is given by the relationship

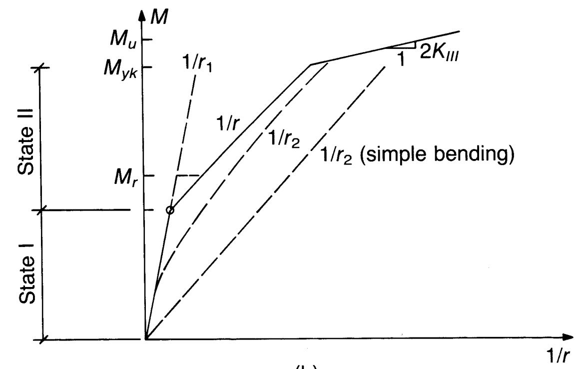
$$\frac{1}{r} = \frac{\varepsilon_{s,m} - \varepsilon_{c,m}}{d} \qquad (3.6\text{–}1)$$

It is also possible to use the diagrams in Figs 3.6.1(a) and 3.6.1(b)



(a)

Fig. 3.6.1(a). Mean curvature—simple bending



(b)

Fig. 3.6.1(b). Mean curvature—bending combined with compression (N = *const.*)

The mean curvature (instantaneous or long-term) in any section of an element may be determined as follows (see Figs 3.6.1(a) and (b)):

$$1/r \;=\; 1/r_1 \quad \text{for state I} \tag{3.6–2a}$$

$$1/r \;=\; 1/r_2 - 1/r_{ts} \;=\; 1/r_2 - (1/r_{2r} - 1/r_{1r})\beta_b(M_r/M) \quad \text{for state II} \tag{3.6–2b}$$

$$1/r \;=\; 1/r_y - (1/r_{2r} - 1/r_{1r})\beta_b(M_r/M_y) + (M - M_y)/2K_{III} \quad \text{for } M \geqslant M_y \tag{3.6–2c}$$

where

$$K_{III} \;=\; \frac{M_u - M_y}{(1/r_u) - (1/r_y)}$$

M is the acting bending moment
M_y is the yielding moment
M_u is the ultimate moment
M_r is the cracking moment

$$M_r \;=\; W_1(f_{ct} - N/A_1) \tag{3.6–3}$$

$1/r_y$ is the curvature corresponding to M_y, N
$1/r_u$ is the curvature corresponding to M_u, N

where

N is the applied normal force,
$f_{ct} = 0.7 f_{ctm}$ if the local deformations are to be considered
$f_{ct} = f_{ctm}$ if the effects of an overall deflection are to be considered
W_1 is the section modulus in state I (including the reinforcement)
A_1 is the section area in state I (including the reinforcement)

$1/r_1$, $1/r_{1r}$ are curvatures in state I corresponding to the action M, N and M_r, N respectively
$1/r_2$, $1/r_{2r}$ are curvatures in state II-naked corresponding to the action M, N and M_r, N respectively

Eq. (3.6–2b) defines a hyperbolic law for the tensioning stiffening effect given by the expression

$$\frac{1}{r_{ts}} = \left(\frac{1}{r_{2r}} - \frac{1}{r_{1r}}\right)\beta_b\left(\frac{M_r}{M}\right)$$

In order to describe the mean behaviour of a structure taking into account previous actions due to shrinkage, temperature, previous loading–unloading of live loads, the mean curvature $1/r$ is assumed to be influenced by cracking already for a smaller value of M than M_r, i.e. for $\sqrt{(\beta_b)}M_r$ for simple bending. This latter value is obtained as the intersection between the descending branch of the curvature $1/r$ and the straight line $1/r_1$, eqs (3.6–2a) and (3.6–2b).

For the case of bending combined with compression, this intersection cannot be expressed by the same simple rule.

For practical applications, numerical values for $1/r_1$ and $1/r_2$ taking into account the reinforcement, creep and shrinkage can be found in CEB Bulletin 158 (Manual on Cracking and Deformations).

$1/r_{ts} = (1/r_{2r} - 1/r_{1r})\beta_b(M_r/M)$ is the tension stiffening

$\beta_b \;=\; \beta_1\beta_2$

β_1 is the coefficient characterizing the bond quality of the reinforcing bars; $\beta_1 = 1$ for high bond bars and 0.5 for smooth bars

β_2 is the coefficient representing the influence of the duration of application or of repetition of loading; $\beta_2 = 0.8$ at first loading and 0.5 for long-term loading or for a large number of load cycles.

However, the possible interaction of shrinkage, creep and relaxation should be appropriately taken into account.

At time t the mean curvature is the sum of the initial (instantaneous) curvature $1/r_0$ and the increment of the curvature $\Delta(1/r)$ due to the time dependent effects (creep and shrinkage of concrete, relaxation of prestressed steel):

$$1/r \;=\; (1/r_0) + \Delta(1/r) \tag{3.6–4}$$

where each of the terms of the second member is evaluated by means of eq. (3.6–2).

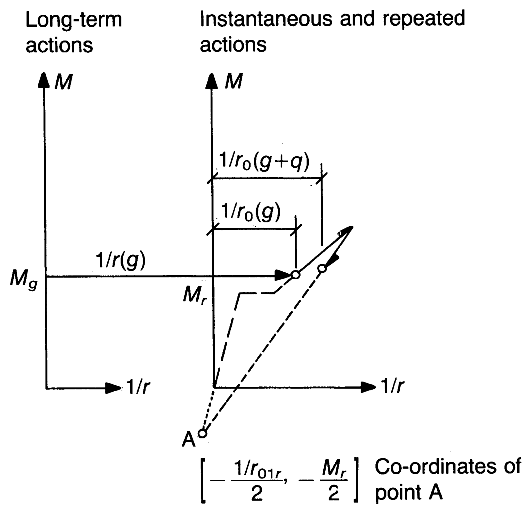


Fig. 3.6.2. Instantaneous mean curvature—simple bending: reloading/unloading

For instantaneous reloading/unloading, the graphical relation given in Fig. 3.6.2 may be taken.

Curvature at time t_2 due to permanent loads g (introduced at time $t_1 < t_2$) and instantaneous loading and unloading q (introduced at time t_2) is given by the following relation (see Figs 3.6.1, 3.6.2)

$$1/r(g+q) \;=\; 1/r(g) + 1/r_0(g+q) - 1/r_0(g) \tag{3.6–5}$$

with

$1/r(g+q)$ is the curvature at time t_2 due to g and q
$1/r(g)$ is the curvature at time t_2 due to g
$1/r_0(g+q)$ is the instantaneous curvature at time t_2 due to g and q
$1/r_0(g)$ is the instantaneous curvature at time t_2 due to g.

3.7. ROTATION CAPACITY

The plastic rotation capacity Θ_{pl} of reinforced concrete flexural members can be derived from the distribution of the average steel strain along the member according to section 3.2. When calculating the distribution of the average steel strain, the shifting of the tensile force by the truss action (see subsection 6.3.3 and clause 9.2.2.5) may be taken into account whenever fully developed shear cracks appear.

$$\Theta_{pl} = \int_0^{l_{pl}} \frac{1}{d - x(a)} [\varepsilon_{sm}(a) - \varepsilon_{sm,y}] da \qquad (3.7\text{–}1)$$

where

l_{pl} is the length of the region, in which the tensile strain is larger than the yield strain
$x(a)$ is the depth of compression zone
$\varepsilon_{sm}(a)$ is the mean steel strain, calculated according to subsection 3.2.3
$\varepsilon_{sm,y}$ is the mean steel strain for $\sigma_s = f_{yk}$
a is the abscissa (see Fig. 3.7.1).

Assuming a bilinear stress–strain relationship for the reinforcement, the plastic rotation capacity Θ_{pl} may be estimated from eq. (3.7–2)

$$\Theta_{pl} = \int_0^{l_{pl}} \frac{\delta}{d - x} \left(1 - \frac{\sigma_{sr1}}{f_{yk}}\right) (\varepsilon_{s2} - \varepsilon_{sy}) da \qquad (3.7\text{–}2)$$

where

δ is the coefficient for taking into account the form of the stress–strain curve of the reinforcement in the inelastic range ($\delta \approx 0.8$)
x is the depth of compression zone
σ_{sr1} is the steel stress in the crack, when the 5%-fractile of the concrete tensile strength is reached
ε_{s2} is the strain of 'naked' bar in the crack
ε_{sy} is the strain at yield stress.

The possible plastic rotation capacity of flexural members may be calculated from the tensile force diagram, which may be determined from the moment distribution; a shifting of the tensile force due to a truss action may be taken into account, if the shear forces are large enough to cause fully developed shear cracks. In this connection an indicative value of the critical V may be taken twice as high as the value V_{Rd1} (see clause 6.4.2.3, eq. (6.4–8)). Using the tensile force–strain diagram of the tensile chord, one gets the strain along the axis of the member. The plastic curvature $(1/r)_{pl}$ integrated over the plastic length l_{pl} gives the plastic rotation capacity Θ_{pl} (see Fig. 3.7.1). $(1/r)_{pl}$ can be calculated from the plastic strain $\varepsilon_{m,pl}$ and the distance $(d - x)$ between the neutral axis and the reinforcing steel.

For the calculation of the steel strains entering eq. (3.7–1), the maximum concrete strain in the critical cross-section will be taken equal to the critical ε_{cu} from eq. (6.2–2) or, in case of adequate confinement, from eq. (3.5–6).

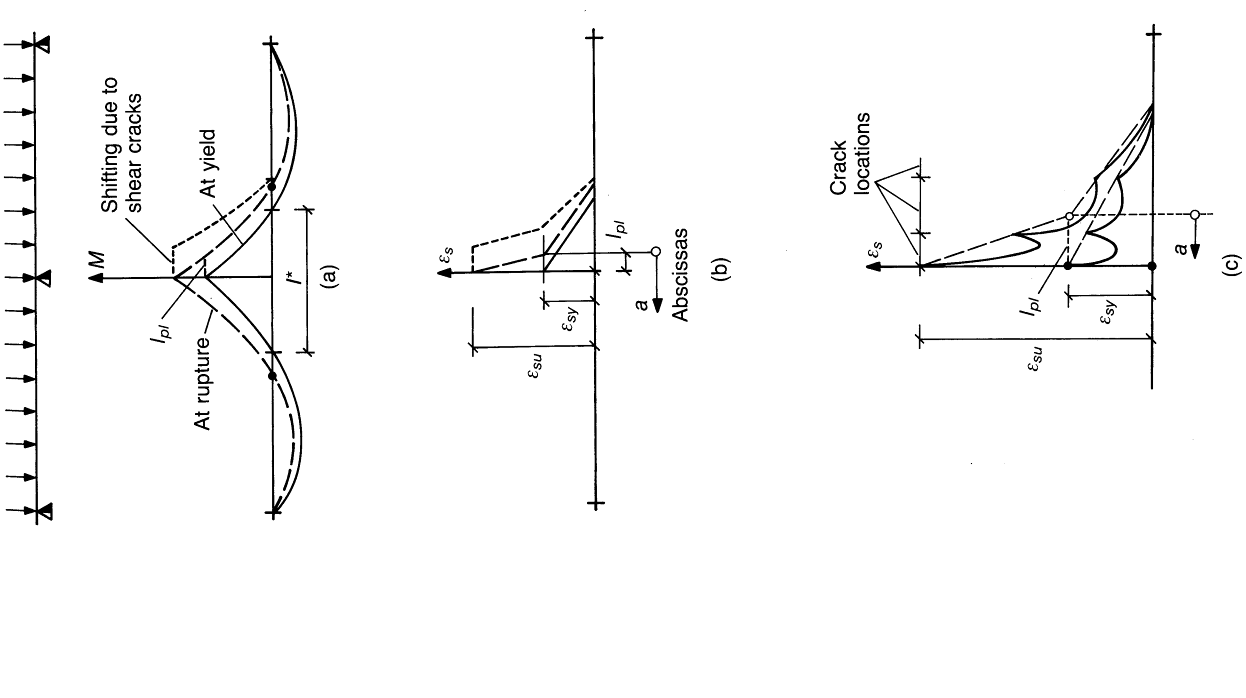


Fig. 3.7.1. Calculation of plastic rotation capacity Θ_{pl}*: (a) flexural moments, M; steel-bar force F* $\cong$ *M/z; (b) steel strain,* ε_s*; (c) detail of steel strain*

In general, in the region of a plastic hinge, a constant height x of the compression zone may be assumed.

In the absence of a more rigorous analysis, the plastic rotation capacity may be estimated by Fig. 3.7.2 instead of eq. (3.7–2).

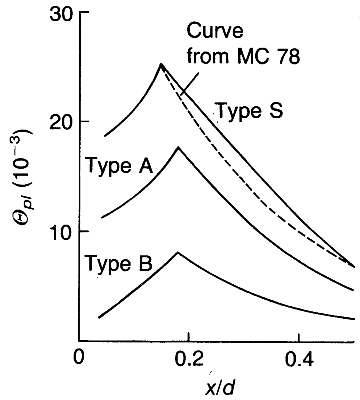


Fig. 3.7.2. Plastic rotation capacity as a functon of the relative depth of the compression zone for deformed non-prestressed bars

(Thus, it gives conservative values for plain bars.)

In order to facilitate practical applications, the abscissas of Fig. 3.7.2 represent the design values of the normalized neutral axis depth (conventionally calculated for f_{cd}, f_{yd}, $\varepsilon_s \leqslant 0.010$, $\varepsilon_c \leqslant 0.0035$ in the critical section).

The maximum plastic rotation capacity occurs at a value x/d, when the concrete strain reaches the value ε_{cu} and the reinforcement is strained to the uniform elongation ε_u (see clause 2.2.4.1). For smaller values of x/d, failure is caused by rupture of the reinforcement; for higher values of x/d by crushing of the concrete in the compressive zone.

For high values of x/d, failure may occur before yielding of the reinforcement. Therefore, the influence of the type of reinforcement on the plastic rotation capacity decreases with increasing values of x/d.

Fig. 3.7.2 has been calculated applying the 5%-fractile of the material parameters for reinforcement and concrete; these values are sufficiently conservative.

Fig. 3.7.2 neglects the favourable influence of transverse reinforcement and of longitudinal reinforcement in the compression zone. The favourable effect of well developed shear cracks is taken into account.

Fig. 3.7.2 is valid for a slenderness $l^*/d = 6$. For other values of l^*/d, the rotation capacity may be multiplied by the factor $\sqrt{(\frac{1}{6}l^*/d)}$ (l^* denotes the distance between two consecutive zero moment points on both sides of a support).

3.8. TORSIONAL STIFFNESS

For the calculation of the action effects, in the absence of more accurate methods, the eqs (3.8–1) to (3.8–3) should be used; they can be taken as constant for each span.

$$K_I = 0.03E_cC/(1 + 1.0\phi) \qquad (3.8\text{–}1)$$

$$K_{IIm} = 0.10E_cC/(1 + 0.3\phi) \qquad (3.8\text{–}2)$$

$$K_{IIt} = 0.05E_cC/(1 + 0.3\phi) \qquad (3.8\text{–}3)$$

These values are valid for SLS as well as for ULS. They are not applicable in the case of hollow box section beams unless differently proved.

The torsional stiffness K for the rotation per unit length is defined as follows

$$d\theta/\mathrm{d}x = T/K$$

In the expression for K_I the factor 0.30 takes account of the non-linear behaviour of concrete before cracking. If necessary the calculation can be carried out for two extreme values.

where

K_I is the stiffness in state I, uncracked
K_{IIm} is the stiffness in state II, cracked
K_{IIt} is the stiffness in state II, torsional and shear cracks
E_c is the modulus of elasticity of concrete
C is the torsional moment of inertia in uncracked stage
ϕ is the creep coefficient to be used for long-term loading.

3.9. CONCRETE-TO-CONCRETE FRICTION

3.9.1. Definitions

The mechanism of shear transfer along a concrete-to-concrete interface which is simultaneously subject to shear and normal compression is called concrete-to-concrete friction.

The general term 'concrete-to-concrete friction' includes also the mechanism of friction along natural cracks which in literature is often referred to as 'aggregate interlock'.

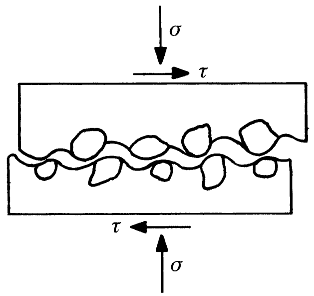


Fig. 3.9.1. Aggregate interlock

The normal compression in the interfaces may be due to one or more of the following actions

- externally acting compressive force (e.g. normal axial load in a column due to vertical loads)
- prestressing force
- clamping effect of tensioned reinforcing bars crossing the interface.

Keyed interfaces are treated in sections 6.10 and 14.3.3.

Concrete faces formed against metal or wooden moulds, as well as concrete surfaces either smoothened after concreting by means of a trowel or without any finishing, are considered as 'smooth'.

Crack interfaces, as well as concrete faces artificially roughened (e.g. washed fresh concrete faces, scrabbled or scraped interfaces) are considered as 'rough'.

3.9.2. Design shear stresses

3.9.2.1. Smooth interfaces

The shear resistance of an interface due to concrete-to-concrete friction may be evaluated by means of the following expression

$$\tau_{fu,d} = 0.40\sigma_{cd} \qquad (3.9\text{–}1)$$

where σ_{cd} is the averaged normal compressive stress on the interface due to external actions and/or prestressing, and calculated taking account of the appropriate γ_F factors, corresponding to favourable effects of permanent and variable actions.

The shear slip needed for the mobilization of $\tau_{fu,d}$ may be calculated as follows

$$s_u = 0.15\sqrt{\sigma_{cd}} \qquad (3.9\text{–}2)$$

where s_u is in mm and σ_{cd} in MPa.

Although the shear resistance of interfaces is due to stresses acting on the regions of contact between the two faces, the design shear stresses given in this section should be considered as average shear resistance of the total area the interface.

When the unfavourable effects of friction are accounted for, the following equation may be used

$$\tau_{fu,d} = 0.60\sigma_{cd}$$

where σ_{cd} is the averaged normal stress acting on the interface. In this case, σ_{cd} should take into account γ_F-values corresponding to the unfavourable effect of permanent and variable actions.

Eq. (3.9–1) denotes the friction between concrete surfaces, which are cast separately.

3.9.2.2 Rough interfaces

The shear resistance of an interface due to concrete-to-concrete friction may be evaluated by means of the equation

$$\tau_{fu,d} = 0.40 f_{cd}^{2/3}(\sigma_{cd} + \rho f_{yd})^{1/3} \qquad (3.9\text{–}3)$$

where

f_{cd} is the design value of the compressive strength of concrete
f_{yd} is the design yield stress of the reinforcement which perpendicularly intersects the interface
ρ is the reinforcement ratio.

The shear stress values given in eq. (3.9–3) correspond to a shear slip value approximately equal to 2.0 mm.

The validity of this equation has been checked for concrete strengths up to 65 MPa, in which the major part of the particles did not fracture during the formation of the rough interface. However, $\tau_{fu,d}$-values cannot be higher than the one which in combination with high normal compressive stresses may lead to global damage of the concrete mass. Since equation (3.9–3) was derived on the basis of tests in relatively small areas of interfaces, it overestimates the shear resistance due to friction of large interfaces. Therefore, in case of large interfaces the $\tau_{fu,d}$ values calculated according to eq. (3.9–3) should be appropriately decreased.

When the unfavourable effects of friction are accounted for, the maximum mobilized friction stress may be calculated according to the following formula:

$$\tau_{fu,d} = 0.65 f_{cd}^{2/3}(\sigma_{cd} + \rho f_{yd})^{1/3}$$

In this case, σ_{cd} is determined taking into account γ_F-values corresponding to unfavourable effects of permanent and variable actions.

Where less than s_u shear slip occurs along the interface, the mobilized shear stress corresponding to the actual shear slip value may be calculated as follows (see Fig. 3.9.2):

for $s < 0.10$ mm

$$\tau_{fd} = 5\tau_{fu,d}\,s \tag{3.9–4}$$

for $s \geqslant 0.10$ mm

$$\left[\frac{\tau_{fd}}{\tau_{fu,d}}\right]^4 - 0.5\left[\frac{\tau_{fd}}{\tau_{fu,d}}\right]^3 = 0.3s - 0.03 \tag{3.9–5}$$

with s in mm.

The shear slip along a rough interface is accompanied by a crack opening (dilatancy), which may be calculated as follows

$$w = 0.6s^{2/3} \tag{3.9–6}$$

with w and s in mm.

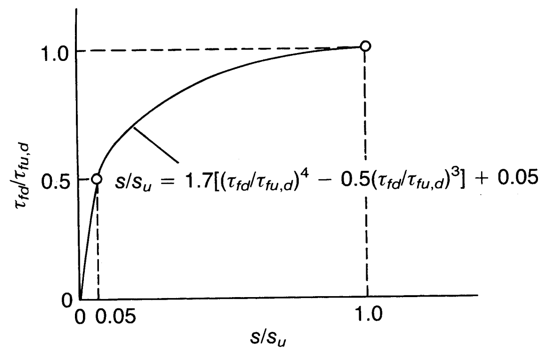


Fig. 3.9.2. Mobilized shear stress τ_{fd} as a function of shear slip s (s_u = *2.0 mm*)

3.10. DOWEL ACTION

This section applies only to dowels which are placed before casting of the concrete.

The design value of the maximum shear force which may be transferred by a reinforcing bar crossing a concrete interface (dowel action) may be calculated by means of eq. (3.10–1), provided that the geometrical conditions, listed below, are satisfied

For smaller concrete covers than according to Fig. 3.10.1, the dowel strength may be neglected, since splitting of the concrete occurs at very small shear displacement values. However, in specific cases, data from available literature may be used for the evaluation of F_{ud}-values corresponding to smaller concrete covers under the condition that adequately small shear displacements are secured and consequences of brittle behaviour are appropriately faced.

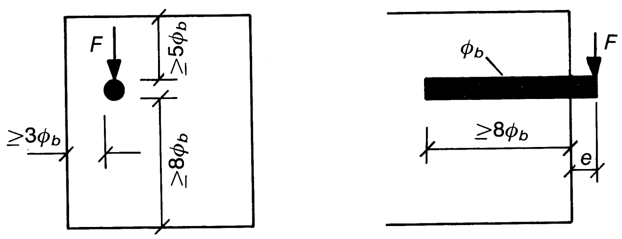


Fig. 3.10.1 Geometrical conditions

$$F_{ud} = \frac{1.30}{\gamma_{Rd}} \phi_b^2 \{\sqrt{[1 + (1.3\varepsilon)^2]} - 1.3\varepsilon\} \sqrt{[f_{cd} f_{yd} (1 - \zeta^2)]} < A_s f_{yd} / \sqrt{3} \quad (3.10\text{–}1)$$

with

$$\varepsilon = 3 \frac{e}{\phi_b} \sqrt{\frac{f_{cd}}{f_{yd}}}$$

where

ϕ_b denotes the diameter of the dowel
A_s denotes the cross-sectional area of the dowel
f_{cd} is the design value of the compressive strength of concrete
f_{yd} is the design value of the steel yield stress
e is the load eccentricity (see Fig. 3.10.1)
γ_{Rd} is the supplementary partial coefficient, may be taken equal to 1.3
$\zeta = \sigma_s / f_{yd}$ (where σ_s is the simultaneous axial stress on the bar).

The shear displacement along a concrete-to-concrete interface, which is needed for the mobilization of F_{ud} may be taken equal to $0.10\phi_b$.

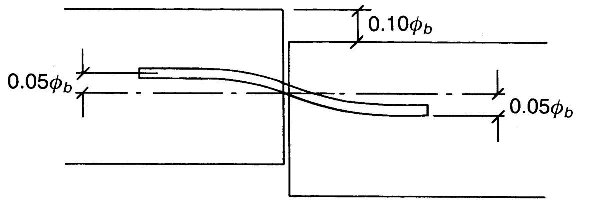


Fig. 3.10.2. Shear displacement needed for mobilization of F_{ud}

4. DATA FOR PRESTRESSING

4.1. TYPES OF PRESTRESSING

The prestress considered in this Model Code is exerted by tendons made of high-strength steel (wires, strands or bars).

Tendons may be used

(*a*) internal to the concrete, and

- (i) pretensioned, or
- (ii) post-tensioned; in this case they may be bonded by grouting, or provisionally or permanently unbonded

(*b*) external to the concrete; they may then be

- (i) totally within the external outline of the structure, or
- (ii) partially or totally outside (except in anchorage points) this outline.

The prestress may be

- — non-detachable and non-adjustable (which is always the case for pretensioning and internal bonded tendons),
- — non-detachable but adjustable,
- — detachable and adjustable.

Anchorages may be active or passive or coupling.

An element of prestressed concrete is considered as fully prestressed if it is designed under restricted tensile stresses under service conditions. Otherwise it is considered as partially prestressed.

4.2. STRESSES AT TENSIONING, TIME OF TENSIONING

The maximum tensile force in the tendons at tensioning should generally not exceed the lower of the following values after transfer or prestressing to the concrete

$$\sigma_{po,\max} = 0.75 f_{ptk} \tag{4.2–1}$$

$$\sigma_{po,\max} = 0.85 f_{p0.1k} \tag{4.2–2}$$

For prestressed bars additional information is needed (e.g. refer to approval documents).

The minimum concrete strength required at the time when tensioning takes place is given in the approval documents for the prestressing system concerned.

For prestressing before placing the concrete (pretensioning) as well as in post-tensioning before transfer of prestressing to the concrete (in the jack) the following limiting values are recommended

$$\sigma_{po,\max} = 0.80 f_{ptk} \tag{4.2–3}$$

$$\sigma_{po,\max} = 0.90 f_{p0.1k} \tag{4.2–4}$$

Where couplers are used, relevant test data and technical approval documents should be consulted.

For prestressing after hardening of the concrete (post-tensioning), unforeseen deviation of frictional behaviour on the site can be important, as it may be impossible to obtain the needed prestressing force under the

limitations of this clause. In such cases it is possible, if the available steels and prestressing technique allow it, to apply a higher stress at the end of the tendons. This stress should never exceed the value of $0.95 f_{p0.1k}$.

To comply with these limitations, special steps may have to be taken in the design of post-tensioned structures.

One solution consists of providing in the design the possibility to insert additional tendons (e.g. in supplementary ducts). Another solution consists of a limitation of the force (within the tendons at tensioning) as a function of the expected immediate losses due to friction according to Table 4.2.1.

Table 4.2.1. Limitation of force at tensioning

For tensioning at one end $\beta = \mu(\alpha + kx)$	$\beta \leqslant 0.28$	$0.28 < \beta \leqslant 0.40$	$0.40 < \beta \leqslant 0.60$
For tensioning at both ends $\beta = \mu(\alpha + kx)$	$\beta \leqslant 0.55$	$0.55 < \beta \leqslant 0.80$	$0.80 < \beta \leqslant 1.20$
$\sigma_{po.max}$	$0.9 f_{p0.1k}$	$0.8 f_{p0.1k}$	$0.75 f_{p0.1k}$

For $\mu(\alpha + kx) > 0.60$ (in the case of tensioning at one end) or 1.20 (in the case of tensioning at both ends) the possibility to insert additional tendons should always be provided for in the design.

For definitions of μ, α, k and x see clause 4.3.3.2.

Regarding the practical aspects of tensioning and grouting refer to sections 11.7 and 11.8.

Where particular rules are not given, the time when prestressing takes place should be fixed with due regard to

- deformation conditions of the component
- safety with respect to the compressive strength of the concrete
- safety with respect to local stresses
- early application of a part of the prestress, to reduce shrinkage effects.

4.3. INITIAL PRESTRESS

4.3.1. General

The initial prestress (at time $t = 0$) is calculated taking into account the prestressing force and the permanent actions present at tensioning.

The value of the initial prestressing force (at time $t = 0$) at a given section of abscissa x, is obtained by subtracting from the force at tensioning the different immediate losses described in this section.

4.3.2. Losses occurring before prestressing (pretensioning)

The following losses should be considered in design

(*a*) loss due to friction at the bends (in the case of curved wires or strands)
(*b*) losses due to drive-in of the anchoring devices (at the abutments) when anchoring on a prestressing bed

(*c*) loss due to relaxation of the pretensioned tendons during the period which elapses between the tensioning of the tendons and prestressing of the concrete.

4.3.3. Immediate losses generally present

4.3.3.1. Losses due to the instantaneous deformation of concrete

Account should be taken of the loss in tendon force corresponding to the deformation of concrete

- *in the case of post-tensioned tendons*, taking into account the order in which the tendons are stressed
- *in the case of pretensioned tendons*, as a result of their action when they are released from the anchorages.

4.3.3.2. Losses due to friction (post-tensioned tendons)

In a cross-section which is at a distance x from an operative anchorage device, the stress $\sigma_{po}(x)$ in the tendon being tensioned is lower than the stress at the anchorage $\sigma_{po,\max}$. The difference between these two stresses corresponds to losses due to friction:

$$\sigma_{po}(x) = \sigma_{po,\max} e^{-\mu(\alpha + kx)} \qquad (4.3\text{–}1)$$

where

μ denotes the coefficient of friction between the tendons and their sheathing

α denotes the sum of the angular displacements over a distance x (irrespective of direction or sign)

k denotes an unintentional angular displacement (per unit length) depending on the design layout (shape) of the tendon.

Values for μ and k are deduced from previous experience with the same type of materials and construction. When values of these coefficients are given in approval documents, they have to be taken as design values.

Before the above values are adopted, however, the diameters of the sheathing, the distance between supports and the quality of workmanship should be duly taken into consideration.

With external prestressing, the friction is concentrated at deviation devices.

All values given below should be considered as indicative mean values.

(*a*) *Internal post-tensioning (with grouting)*
The coefficient of friction μ is the product of the physical coefficient of friction μ_o and the squeezing factor χ, where χ is dependent on the degree of filling of the duct. Where more exact investigations are not available, this factor can be assumed to be 1.3 to 1.35 for tendons filling the duct between 50% and 60%. The physical coefficient of friction μ_o is influenced, interalia by the surfaces of prestressing steel and ducts (micro- and macro-structures), rust, pressure, elongation of the tendon, etc.

If more accurate values are not available and in the case of steel and duct being both without rust, the values given below can be assumed, for μ_o and for μ with a 50% filling. These values which are indicative mean values can be multiplied by 0.9 if slight lubrication is present, e.g. by means of soluble oil.

Cold drawn wire	$\mu_0 = 0.13$	$\mu = 0.17$
Strand	$\mu_0 = 0.15$	$\mu = 0.19$
Deformed bar	$\mu_0 = 0.50$	$\mu = 0.65$
Smooth and round bar	$\mu_0 = 0.25$	$\mu = 0.33$

Under site conditions variations of 50% are possible. In the case of rust, even higher variations are likely to occur.

The coefficient k takes account of unintentional angular deviations. Its value depends on the quality of workmanship and on the distance between supports of the tendon. Values for k are given in approval documents with $k = 0.005$–0.01 (m^{-1}).

For the verification of the real values of prestressing losses at tensioning (see clause 11.7.3.2(d)) it is recommended to measure the transmission of prestressing force from one end of the tendon to the other.

(*b*) *Friction losses in the case of unbonded internal tendons*
Tests and practical experience have shown that the friction factors μ and k as listed below can be applied.

- For monostrands (individual strands in plastic; single or grouped) in slabs or reservoirs $\mu = 0.05$–0.07 $k = 0.006$–$0.01\ m^{-1}$
- For greased multistrand or multiwire tendons (e.g. in nuclear containments) $\mu = 0.13$–0.15 $k = 0.004$–$0.008\ m^{-1}$
- For dry multistrand or multiwire tendons (e.g. in nuclear containments with dry air as subsequent corrosion protection) factors as for ordinary post-tensioned tendons.

(*c*) *In the case of external multistrand tendons*

- For naked dry strands over steel saddle $\mu = 0.25$–0.30 $k = 0$
- For greased strands over steel saddle $\mu = 0.20$–0.25 $k = 0$
- For dry strands inside plastic pipe over saddle $\mu = 0.12$–0.15 $k = 0$
- For bundle of monostrands (individual sheets in plastic) over saddle $\mu = 0.05$–0.07 $k = 0$

These values correspond to a saddle radius of 2.5 to 4.0 m. For lower radii further test evidence is needed.

4.3.3.3. Losses caused by recoil of the steel (post-tensioned tendons)

Account must be taken of the loss which occurs when there is a drive-in of the tendons at the anchorage, during the operation of anchoring after tensioning, and of the deformation of the anchorage.

The values to be taken into consideration are defined in the approval documents for the prestressing system concerned.

Due to this drive-in, the highest stress within the tendon is no longer at the anchorage.

4.3.3.4. Effect of heat-curing (pretensioning)

Two types of losses have to be taken into account

- reduction of steel stress due to an acceleration of relaxation during heat treatment
- direct thermal effect.

The loss of prestress due to relaxation during the heat treatment can be equated to 75% of the total value of relaxation losses.

(a) Relaxation losses
Relaxation losses can be calculated by adding to the value of time a duration defined by the formula

$$t_{ep} = t_{p1}\, 1.04^{(T_{max}-20)} \qquad (4.3\text{–}2)$$

where

T_{max} is the maximum temperature of the concrete during heat treatment in °C,
t_{p1} is the mean duration of the heating cycle, calculated by eq. (4.3–3):

$$t_{p1} = \frac{1}{T_{max} - 20} \int_o^{t_1} [T(t) - 20]\, dt \qquad (4.3\text{–}3)$$

where

t_1 is the age of the concrete when its temperature returns to ambient temperature
$T(t)$ is the temperature of concrete, in °C, at time t.

(b) Losses of direct thermal origin
Direct thermal effect is caused by

- the dilatation of concrete, when it is not bonded to the prestressing steel
- the restraint to the dilatation of concrete presented by the prestressing steel when it is bonded.

This type of loss does not exist with moulds supporting the tension of tendons and heated together with concrete.

The losses of direct thermal origin can be calculated by eq. (4.3–4)

$$\Delta\sigma = 0.9 E_p \alpha_p (T_{max} - T_0) \qquad (4.3\text{–}4)$$

where

E_p is the elastic modulus of steel
α_p is the coefficient of thermal expansion of steel
T_0 is the temperature of steel at tensioning
T_{max} is the maximum temperature of steel during heat curing.

4.3.3.5. Other immediate losses

Account should be taken of all possible causes of immediate loss of tension related to the tensioning process or the equipment used for prestressing.

4.4. VALUE OF PRESTRESSING FORCE

The initial prestressing force in a tendon is the force existing in this tendon at the end of the prestressing operation. The initial prestressing force on a prestressed element is obtained by considering all the forces existing in the tendons, at the end of the last prestressing operation.

The prestressing force at a given time t is obtained by subtracting from the initial prestressing force the value of the time dependent losses at this time t.

These losses are due to creep and shrinkage of concrete and relaxation of steel.

The final value of the prestressing force is obtained by subtracting from the initial prestressing force the maximum expected value of the time-dependent losses.

4.4.1. Calculation of time-dependent losses

The time-dependent losses are calculated by considering the following two reductions of stress:

(*a*) the reduction of stress, due to the reduction of strain, caused by the deformation of concrete due to creep and shrinkage, under quasi-permanent actions:

(i) for bonded tendons, the local deformation at the level of the tendon has to be considered;
(ii) for unbonded tendons, the deformation of the whole structure

Data for calculation of the deformations of concrete under creep and shrinkage are given in section 2.1.

Ordinary reinforcement has an influence on the value of time-dependent shortening of concrete.

The interaction can be estimated as described in CEB Bulletin 199.

between the constraints of the tendons has to be taken into account;

(*b*) the reduction of stress within the steel due to the relaxation of this material under tension.

Data on the relaxation of steel are given in section 2.3.

The relaxation of steel depends on the reduction of strain due to creep and shrinkage of concrete. This interaction can generally be neglected.

The reduction of strain in steel due to time-dependent losses may be calculated by dividing the stress loss by the modulus of elasticity of steel.

4.5. BOND PROPERTIES OF POST-TENSIONED TENDONS

4.5.1. General

The main representative elements of the bond of a tendon are the ultimate value of bond stress, and the relationship between the bond stress and the value of the relative slip between concrete and tendon.

Values of design bond stress are given in subsection 6.9.10.

Bond stress and slip are measured at the external surface of the sheathing, but the bond values depend on

- the nature of the prestressing steel: wires, strands, bars
- the structure of the surface of these elements.

Sheathing can be made of steel or plastic; it can be corrugated or smooth.

The origin of variations of deformation is the reference state of decompression at the level of the tendons.

4.5.2. Numerical values

The bond properties of post-tensioned tendons can be given by comparison with bond properties of ribbed bars of same diameter, used for ordinary reinforcement. For a given value of bond slip, the value of bond stress for a post-tensioned tendon can be deduced from the bond value of an ordinary bar by a reduction factor η_p.

Calculation of the distribution of internal stresses is necessary for verification under fatigue effects.

The following values of η_p can be adopted for the calculation of crack widths and of the distribution of internal stresses

η_p = 0.2 for smooth prestressing steels
η_p = 0.4 for strands
η_p = 0.6 for ribbed prestressing steels.

These values of η_p are established on the basis of tests made on tendons with a corrugated steel sheathing, and a maximum diameter of 56 mm.

These values for η_p for grouted tendons, are different from the values given for direct bond strength, see clause 6.9.11.2.

In the general case when ordinary reinforcement is present together with post-tensioned tendons in the same section, the calculation of probable crack widths should be made taking into account the different diameters and bond properties of the reinforcing elements.

Tensile stresses are accepted in partially prestressed structures, where ordinary and prestressing reinforcement are used simultaneously.

4.6. DESIGN VALUES OF FORCES IN PRESTRESSING TENDONS

4.6.1. General

Prestress is usually exerted by one set of tendons. The total permanent force exerted at a given section (abscissa x), and at a time t, by the whole set is considered as the prestressing force; in exceptional cases, several prestressing forces (practically never more than two), should be considered separately. These cases should be identified by judgement. The criteria, to be simultaneously satisfied, are that

- the effect of the two sets are of contrary senses
- these effects have the same order of magnitude
- the dispersions are relatively high and there are qualitative reasons why they should not be considered as correlated.

4.6.2. Definition of prestress

Depending on the calculation, prestress is represented by various physical quantities, called indicators, each of them corresponding to a reference state. These reference states are specified in the relevant clauses.

If the prestressing steel remains elastic, the prestressing force $P(x, t)$ and the prestressing strain $\varepsilon(x, t)$ are connected by the relation $\varepsilon(x, t) = P(x, t)/E_p A_p$. In general this relation should be supplemented by a term due to the relaxation of the steel.

For example

- For the calculation of losses and the structural analyses the indicator is the prestressing force or stress at a time for a given section (x) when the structure is subjected to permanent actions (without γ factors) or, more precisely, to the quasi-permanent combination of actions
- The hyperstatic prestress is a complementary indicator deduced from the indicator defined above (see clause 1.4.3.1.)
- For some verifications of resistance (when the yield strength of prestressing steels is exceeded) the indicator is the difference of strain between tendons and the adjacent concrete (see OB on Fig. 1.4.1).

Other indicators, e.g. the strain corresponding to a zero curvature, have been used in some background studies. An analysis of various indicators is given in CEB Bulletin 202.

Whatever the selected indicator is, a mean value of prestress is defined. Two characteristic values (an upper and a lower) are also defined, taking into account the possible variations of the losses and of the tensioning force.

The characteristic values are normally calculated by means of the approximate formulae given in clause 1.4.3.2. If the field of application of these formulae is infringed, for instance in the case of very large immediate losses, more precise formulae have to be used, such as

$$P_{k,\sup}(k, t) \;=\; P(0, 0) - 0.7[\Delta P(x, 0) + \Delta P(x, t)]$$

$$P_{k,\inf}(k, t) \;=\; P(0, 0) - 1.30[\Delta P(x, 0) + \Delta P(x, t)]$$

4.6.3. Design values for SLS verifications

For all verifications relating to cracking (decompression included), for the analysis of the fatigue effect and in specific cases where the size of the prestress influences the result in a large overproportional way, the less favourable characteristic value should be taken into account.

It is only acceptable to introduce a mean value P_m for deformation calculations, and for the verification of limiting compressive stresses. See chapter 7.

4.6.4. Design values for ULS verifications

For the verifications with regard to fatigue and, if relevant, to static equilibrium, the less favourable characteristic value should be taken into account.

For the other ultimate limit states a factor $\gamma_{P\,\mathrm{sup}}$ or $\gamma_{P\,\mathrm{inf}}$ shall be applied in accordance with clauses 1.4.3.2. and 1.6.2.4.

Depending on the type of structural analysis the γ_P factors may have to be introduced at the level of the input of the analysis or at the level of the output (i.e. applied to the effects of P), or even partially at each level (see clause 1.6.2.4(c)). In many cases, $\gamma_{P,\mathrm{inf}}$ being equal to 1.0, no practical problem is met. In other cases, e.g. for the assessment of hyperstatic effects or where the yield strength of the prestressing steel is exceeded, γ_P may be taken equal to 1.0 and P may be taken equal to P_m, as acceptable approximations.

4.7. ANCHORAGE AND COUPLING OF PRESTRESSING FORCES (POST-TENSIONING)

4.7.1. General

After hardening of the concrete, the tendons are tensioned and their extremities are fixed within anchorages, which transfer the prestressing forces to the concrete.

When cement grouting is applied, the transmission of the prestressing force is not dependent on the anchorages only.

Coupling devices, or couplers, may be used to connect the end of a tendon, which is tensioned first, to the end of a second tendon, placed as a prolongation of the first, and which will be prestressed in a second stage.

With unbonded tendons, special attention should be given to the fact that the anchorages are the only means of transmission of the prestressing force to the structure. It may be necessary to place intermediate anchorages, functioning in both directions, to prevent the risk of progressive collapse, when the strength of the structure is achieved by one set of tendons extending over many spans.

Information relating to anchorage arrangements are given in the approval documents. When the assumptions or service conditions differ from those envisaged by these, additional experimental checks may be necessary.

The deviators have to be designed under the unfavourable assumption that a relative displacement of the tendon takes place, resulting in friction on the deviation device. Generally the calculation of the structure itself can be made assuming the tendons are fixed at the deviating points.

With external prestressing, deviating devices are placed between the tendons and the structure. These devices, and their fixing zones, have to be designed to transfer the corresponding design action, taking the permissible tolerances into account.

With external prestressing, it is recommended to provide for the replacement of the prestressing tendons.

4.7.2. Transfer of load from the tendon-anchorage-assembly to the concrete

The strength of the anchorage zones should exceed the characteristic strength of the tendon, both under static load and under slow-cycle load.

Possible formation of small cracks in the anchorage zone may not impair the permanent efficiency of the anchorage if sufficient transverse reinforcement is provided.

This condition is considered to be satisfied if stabilization of strains and cracks widths is obtained during testing.

4.8. CORROSION PROTECTION OF TENDONS

4.8.1. General

The principles of corrosion protection of tendons in the temporary and permanent state are given in chapters 8 and 11.

PART II. DESIGN PROCEDURES

5. STRUCTURAL ANALYSIS

5.1. GENERAL

Structural analysis is defined as the determination of the action effects over the whole or part of a structure, with the purpose of carrying out a verification at the ultimate and serviceability limit states.

This chapter is established in accordance with the concept and rules defined in sections 1.3 and 1.6. Unless stated otherwise, the numerical data for applying it are the design values of the basic variables as defined in sections 1.4, 1.6, 4.6 and 6.2. In some cases, e.g. when using FEM (or also in the cases of clauses 1.6.2.2 and 1.6.3.2), the fulfilment of some verification conditions may be obtained in the course of the analysis itself.

Structural analysis means an overall analysis as defined in section 1.3.

The considered action effects may be

- stresses (see eq. (1.3–3)).
- sectional forces (axial and shear) or moments (bending and torsional) (see eq. (1.3–2))
- geometrical quantities (deflections, rotations, crack widths) (see eq. (1.3–4))
- vibrations, however these are generally indirectly treated by specific methods (see section 7.6).

Except for the determination of stress resultants in isostatic structures, no structural analysis can be carried out without assumptions about material behaviour.

5.2. IDEALIZATION OF THE STRUCTURE

5.2.1. Dimensional classification of structural elements

For the analysis the structural elements are classified as

- one-dimensional when one dimension is much larger than the other two
- two-dimensional when one dimension is relatively small compared with the other two
- three-dimensional when no dimension is largely prevailing.

Columns, beams, arches are considered one-dimensional elements, if the length is larger than three times the overall sectional depth.

Slabs, plates, deep beams, walls, shells are considered two-dimensional elements. Slabs and deep beams are defined as

- slabs, if in presence of transverse forces the minimum distance between adjacent regions of zero moments is not less than four times the overall thickness
- deep beams, if the span is less than twice the overall depth.

The definition of structural schemes could be made with reference to the state of stress rather than to geometrical dimensions. According to this approach the definitions are

- one-dimensional schemes when normal stresses in one direction predominate over those in the orthogonal directions
- two-dimensional schemes when normal stresses in two orthogonal directions predominate over those in the third
- three-dimensional schemes when normal stresses do not predominate in any of the three orthogonal directions.

5.2.2. Classification in terms of level of discretization

The methods of analysis may refer to

- cross-sections of structural members
- a fibre along a structural member
- finite elements.

5.2.3. Geometrical data

5.2.3.1. Effective width of flanges

In T-beams a uniform distribution of longitudinal stresses over a reduced width of the flange, called effective width, may be assumed for all limit states, both for the overall analysis and for section verification.

The effective width may be determined on the basis of calculations by elastic or plastic theory, and may be varied along the axis of the beam.

In the absence of a more accurate determination, the effective width to be used in the overall analysis should be taken equal to the thickness of the web, plus 1/5 of the approximate distance between the points of zero moment, but not exceeding the actual width of the top slab. In this approximation the effective width can be taken as constant over the entire span, including the parts near intermediate supports for continuous beams.

For edge beams the effective width is taken equal to the thickness of the web plus 1/10 of the distance between the points of zero moment.

The points of zero moment considered above may practically be considered as fixed for all calculations and associated with the distribution of moments due to the permanent weight.

5.2.3.2. Effective span

Usually, the span l has to be introduced as the distance between adjacent support axes. When reactions are located significantly away from the axis of support, the effective span has to be calculated taking into account the real position of the support section.

In the overall analysis of frames, when l_{ef} is less than the distance between axes of columns the dimensions of the joints should be taken into account by introducing rigid elements between the centroidal axis of the column and the end section of the beam.

5.2.3.3. Cross-sections

The gross concrete section is defined as the total concrete section, where the contribution to structural stiffness of the reinforcement and the area of ducts for prestressing tendons is disregarded. If these are taken into account, by adopting a suitable modular ratio for the reinforcement, the section is designated as the transformed concrete section.

5.3. CALCULATION METHODS

5.3.1. Basic principles

Any structural analysis shall satisfy the equilibrium conditions.

If not otherwise stated, the compatibility conditions shall always be satisfied in the limit states considered.

In the cases where verification of compatibility is not directly required, suitable ductility conditions should be satisfied and adequate performance under service conditions be ensured. In general the equilibrium conditions are formulated for the undeformed system (first order theory).

In case of slender structures as defined in clause 1.6.3.1 the equilibrium shall be verified for the deformed system (second order theory). In these cases the influence of deformations on action effects shall be considered.

5.3.2. Types of structural analysis

The choice of method depends on the assumed behaviour of the materials and the possible consideration of the effects of deformation on the action effects (second order effects).

The overall analysis of a structure can be performed according to the following methods

- non-linear analysis
- linear analysis
- linear analysis with redistribution
- plastic analysis.

In this context plastic analysis is meant as both upper bound and lower bound solutions.

5.3.2.1. Non-linear analysis

The structural analysis is defined as non-linear if a non-linear behaviour is assumed for the materials. Second order effects may or may not be taken into account.

Equilibrium and compatibility conditions should be fulfilled.

This approach implies that non-linearity arising from a realistic consideration of the structural behaviour should be taken into account in the 'response relationship' (non-linear constitutive laws for materials, non-linear force deformation relationship for cross-section members or sub-assemblages, due to material properties, cracking and second order effects).

The method generally requires an initial definition of the geometry of the structure and of the reinforcement.

The non-linear behaviour of materials and the second order effects are considered by suitable incremental and/or iterative numerical methods.

Non-linear analysis is a realistic description of the physical behaviour and therefore a method completely consistent with the assumptions used for the local verification and member design; it should be used as a reference for other more simplified approaches.

5.3.2.2. Linear analysis

The structural analysis is defined as linear if a linear elastic behaviour is assumed for the materials.

Concerning the cross-section to be taken into account, see clause 5.2.3.3.

This approach implies that the 'response relationship' is linear, and the assumption of reversible deformations is retained. The results are realistic only under the condition that actions are low and members are uncracked.

For ULS verifications existing practice allows the use of linear analysis without direct verification of sufficient ductility. This is based on the assumption that there is ductility enough to balance the lack of compatibility, under the conditions stated in subsection 5.4.2 for beams and frames.

The method is normally used with the gross-section of concrete members; therefore it requires definition of geometry of the structure, but not necessarily of the reinforcement.

Cracked cross-sections may, however, be used if in the limit state under consideration a fully developed crack pattern can be expected.

The results of a linear analysis are also used in the verification for the serviceability limit state.

5.3.2.3. Linear analysis with redistribution

The structural analysis is defined as linear with redistribution, if the action effects derived from a linear analysis are redistributed in the structure. Equilibrium and suitable ductility conditions should be satisfied.

Although the highest redistribution is obtained in the ULS, even some redistribution takes place in SLS due to cracking.

5.3.2.4. Plastic analysis

The structural analysis is defined as plastic if one of the two basic theorems of plasticity upper bound or lower bound is met. Rigid-plastic or elastic-plastic behaviour for the materials may be assumed.

Plastic analysis is allowed only if sufficient ductility for the attainment of the assumed configurations is ensured.

The method is not allowed when consideration of second order effects is required.

5.4. BEAMS AND FRAMES

5.4.1. Non-linear analysis

5.4.1.1. General

Non-linear analysis may be used both for SLS and ULS verifications under the conditions described in clause 5.4.1.4.

Non-linear analysis can be performed at different levels of complexity according to

- the models of ULS to be considered in the verification
- the model of one-dimensional element to be used in analysis
- the type of constitutive law to be adopted in the analysis and verification of sections
- the type of load history to be adopted in analysis.

5.4.1.2. Models for ultimate limit states

Three basic models can be considered.

- *Model* (*a*) represents 'First Yield', corresponding to the load level at which plastic behaviour takes place for the first time at a critical section.

  For the moment–curvature relationship it is often sufficient to adopt a bilinear representation to describe

  — state 1: linear elastic, uncracked
  — state 2: cracked.

  The conditions of compatibility may be obtained by deducing the curvature of each section from an appropriate moment–curvature relationship and by integrating this curvature to obtain the deformed shape.

  For most construction works and design situations a holonomic structural behaviour may be assumed (see clause 5.4.1.4) and therefore normally a law of deformability (e.g. a moment–curvature law) can be applied assuming monotonic conditions.
- *Model* (*b*) represents the formation of a mechanism which can take place whenever critical sections are ductile enough to permit the formation of this mechanism.
- *Model* (*c*) represents local rupture due to exceedance of the limit plastic rotation θ_{pl} at a critical section.

Models (b) and (c) may be used only for verifications with regard to the ultimate limit state of resistance.

In case a) no evaluation of plastic behaviour of materials is needed in performing the analysis, which takes into account cracking and second order effects only. This approach, which may be called 'non-linear elastic' analysis, should be adopted whenever evaluation of plastic behaviour is uncertain, due to the brittle nature of the failure mechanism.

For models b) and c) a trilinear representation of the moment–rotation relationship can be adopted which also describes state 3 (plastic conditions), characterized by the plastic rotation θ_{pl}.

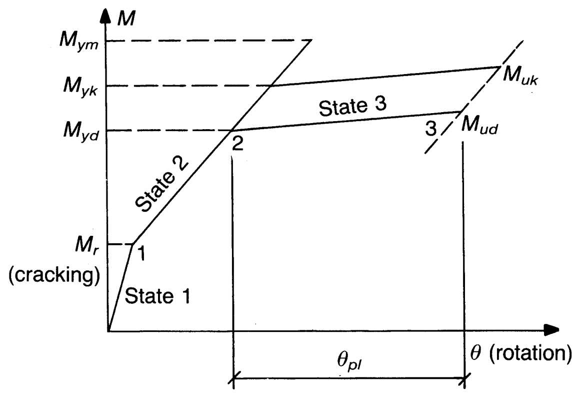


Fig. 5.4.1. Idealized trilinear diagram

Notation for Fig. 5.4.1

M_r is the cracking moment
M_{ym} is the moment corresponding to the yielding of the steel, based on mean values of material properties
M_{yk} is the moment corresponding to the yielding of the steel, based on characteristic values of material properties
M_{yd} is the moment corresponding to the yielding of the steel, based on design values of material properties
M_{uk} is the characteristic value of resisting moment
M_{ud} is the design value of resisting moment.

The third branch is derived assuming an affinity with respect to the characteristic values.

The plastic rotation may be assumed to be concentrated at the critical section; the permissible local plastic rotation θ_{pl} can be obtained from Fig. 5.4.2. Additional rotation capacity can be obtained considering the favourable influence of possible confining reinforcement (see section 3.5).

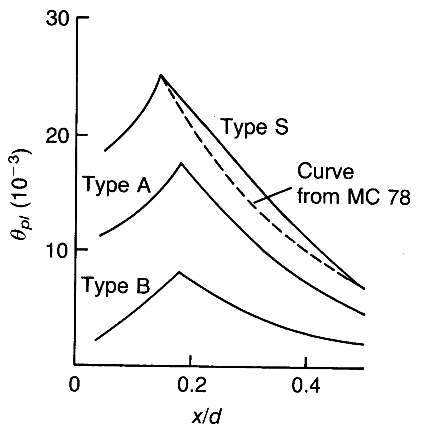


Fig. 5.4.2. Rotation capacity θ_{pl} versus x/d *relative depth of the neutral axis of the critical cross-section at ULS for various steel classes (the* x*-values are conventionally calculated considering design values of material properties, and nominal limit values of $\varepsilon_c = 0.35\%$ and $\varepsilon_s = 1.0\%$)*

With some exceptions, the hypothesis that plane sections remain plane is assumed to be valid.

This approach should be adopted whenever secondary effects such as bond slip, aggregate interlock, and dowel effect, are considered for an adequate modelling of the structure.

5.4.1.3. Models of one-dimensional elements used in analysis

The following models may be used in non-linear analysis of one-dimensional structures

- one-dimensional models, where cracking is considered as diffused along the elements while plasticity is considered as concentrated at critical sections by the introduction of 'plastic hinges', at an appropriate load level.
- one-dimensional 'layered' models, where the state of stress in each layer is derived from the state of strain using the constitutive laws of materials and the element stiffness is defined accordingly
- assemblages of one- and two-dimensional finite models, to model the behaviour of the one-dimensional member at 'micro' level.

5.4.1.4. Constitutive laws for overall analysis and verification of sections

A non-linear analysis for the ULS may be carried out assuming mean values of the material properties up to the level of design-yield stress in the reinforcing steel; after yield is attained in critical regions, design values of the material properties should be used both for analysis and for resistance determination.

As an approximation, material properties associated with the specified characteristic strength of the materials may be used.

Where no major unloading and reloading of the structure is considered, or the plastic behaviour of the structure is not studied as in non-linear elastic analysis, holonomic constitutive laws may be used.

In the other case the non-holonomic nature of the constitutive laws shall be taken into account.

A constitutive law is defined as holonomic when loading and unloading paths are coincident.

5.4.1.5. Type of load history to be adopted in analysis

Two basic types of load history may be adopted whenever an incremental approach is used for non-linear analysis

- proportional, where all loads are incremented using the same multiplying factor
- non-proportional, where the various types of loads are incremented according to a given load history which follows, as far as possible, the order according to which real loads are likely to be applied to the structure.

Whenever an iterative method of analysis is used it should be assumed that the load history is proportional.

Following this approach, it is generally recommended that the calculations be performed in the order

- Apply permanent and quasi-permanent loads ($G + \Psi_2 Q_k$).
- Consider creep and apply the indirect actions with their design values.
- Finally apply progressively the remaining part of the design combination of loads (i.e. $(\gamma_G - 1)G + (\gamma_Q - \Psi_2)Q_k + \ldots$) increasing it proportionally up to the simulated collapse of the structure.

If relevant, due attention should be paid to different stages of construction of structural parts for the same building. When investigating load histories of this type it is relevant to consider the state of internal damage which may have occurred before the load path under consideration. Previous damage may lead to anisotropic constitutive laws.

5.4.2. Linear analysis

Linear analysis is to be applied mainly for serviceability limit states and may also be used for verifying the ultimate limit state for continuous beams and non-sway frames.

At the ULS, a linear analysis cannot always satisfy the conditions of compatibility in view of the invalidity of the assumptions relating to the corresponding deformations. The beams shall be capable of sufficient plastic rotation to prevent local rupture before the calculated moment distribution has been attained.

In the absence of more detailed information, this criterion may be assumed to be met if, for regular orthogonal frames with approximately equal geometry in all spans and storeys, the equivalent slenderness ratio λ^*, according to eq. (5.4–1), does not exceed 30.

For a single isolated column having uniform cross-section, the equivalent slenderness ratio is defined as

$$\lambda^* = \frac{\sqrt{\nu_{sd}}}{1 + 15\rho}\,\lambda \tag{5.4–1}$$

where

$\lambda = l/i$ is the Eulerian slenderness (see also clause 6.6.1.3)
ρ is the geometrical percentage of total longitudinal reinforcement, $= A_s/A_c$
ν_{sd} is the relative axial force $= N_{Sd}/(A_c f_{cd})$.

For a frame, the most unfavourable column with regard to loading and slenderness ratio λ should be checked. Where $\lambda^* > 30$, a more rigorous structural analysis should be carried out according to section 6.6.

The consideration of the equivalent slenderness is an application of the concept that the reinforcement ratio considerably affects second order effects (clause 6.6.3.1.1). When the more stringent conditions of clause 6.6.3.1.1 are not met, the above expression for λ^* may help to avoid a rigorous analysis.

For continuous beams and beams belonging to non-sway frames, sufficient ductility can be assumed to be present, if the relative depth of the neutral axis x/d in the critical cross-section in the ULS is in accordance with the following:

Steel type	*Concrete grades*	x/d
S and A	C12 to C35	$\leqslant 0.45$
S and A	C40 to C80	$\leqslant 0.35$
B	C12 to C80	$\leqslant 0.25$

The ductility is increased by transverse reinforcement. x/d can be reduced by means of suitable compression reinforcement.

In the absence of more precise information on the influence of the type, quality and bond properties of prestressing steels, post-tensioned steel may be assimilated to type A steel, and pretensioned steel to type B steel.

It can also be used for determining the first order loading effects for sway frames, provided that the reduction in bearing capacity due to second order effects does not exceed the limit defined in clause 6.6.1.3.

5.4.3. Linear analysis followed by limited redistribution

5.4.3.1. General

For the verification of ultimate limit state, it is allowed to reduce the moments in the sections subjected to the highest action effects, resulting from a linear analysis, provided that in the other sections the moments are increased to maintain equilibrium.

For a structure subject to various load cases, only one redistribution can generally be assumed.

These rules are mainly meant to be applied when type S and A steels are used.

In principle, all the consequences of the assumed redistribution and of the possible scattering of its value, should also be taken into account in the verification procedure, concerning shear, anchorage of the bars and cracking. In particular, the length of the reinforcing bars shall be sufficient to prevent any other section becoming critical.

In curved beams, flexural yielding may produce a sudden increase of torsion, which can lead to a brittle failure before the redistribution of flexural moments is fully exploited.

5.4.3.2. Ductility conditions

The reduction coefficient δ to be used for multiplying the moments in the sections subjected to the highest moments should satisfy the following conditions to be applied to beams with straight axes in horizontal plane only.

Steel type	*Concrete/structure*	*Reduction coefficient*
S and A	Concrete grades C12 to C35	$\delta \geqslant 0.44 + 1.25x/d$
S and A	Concrete grades C40 to C60	$\delta \geqslant 0.56 + 1.25x/d$
S and A	Continuous beams and non-sway frames	$0.75 \leqslant \delta \leqslant 1.00$
S and A	Sway frames	$0.90 \leqslant \delta \leqslant 1.00$
B	Concrete grades C12 to C60	$\delta \geqslant 0.75 + 1.25x/d$ $0.90 \leqslant \delta \leqslant 1.00$

A redistribution of 25% may result after cracking, owing to the reduction in stiffness due to cracking between zones in the span and over the supports. Such a redistribution need not always be linked to a ductility condition and it does not always take place in the desired direction. The given ductility conditions cover the most unfavourable cases.

This is allowed only if the equivalent slenderness ratio λ^* does not exceed 15.

Post-tensioned steel may be assimilated to type A steel and pretensioned steel to type B steel.

5.4.4. Plastic analysis

It should be verified that the required plastic rotations in the plastic hinges, for the assumed mechanism, are less than the limiting plastic rotations θ_{pl} given in Fig. 5.4.2.

The method is not allowed for sway frames, nor if type B steel is used.

See also comments on clause 5.3.2.4.

5.4.5. Second order effects

In general, second order effects are to be taken into account only for the ultimate limit state.

When second order effects considerably influence the action effects (e.g. sway frames) they should be considered also for the serviceability limit states.

5.5. SLABS

5.5.1. Scope

Section 5.5 applies to solid slabs subjected to two-dimensional bending with or without prestressing. Non-solid slabs such as ribbed slabs, hollow-core slabs etc. are included provided that their behaviour, especially their stiffness, may be simulated by a corresponding solid slab.

5.5.2. Types of analysis

Slabs may be analysed by each of the following methods

- linear analysis
- linear analysis with redistribution
- plastic analysis
  - — plastic analysis considering only equilibrium conditions at the ultimate state of the load carrying capacity of slabs (lower bound solutions), using the static method
  - — plastic analysis assuming a yield line collapse mechanism for the ultimate state (upper bound solutions), using the kinematic method
- non-linear analysis.

The actual behaviour of slabs under loads and imposed deformations is covered best by a non-linear analysis which may be regarded as the reference solution, whereas linear analysis is covering primarily the serviceability and plastic analysis primarily the ultimate limit state.

This may be carried out by adopting non-linear stress–strain relations or non-linear moment–curvature relations satisfying conditions of equilibrium as well as compatibility.

5.5.3. Linear analysis

The method is based on the theory of elasticity adopting a linear moment–curvature relationship and a value for Poisson's ratio between 0.0 and 0.2.

If the reinforcement is determined on the basis of this method, it is not necessary to verify rotation capacities. Moments with local steep gradients (concentrated loads etc.) may be distributed in a convenient wider area provided equilibrium is preserved.

With regard to limit analysis the solution of the linear theory of elasticity is a lower bound solution.

In practice, for the usual cases, the results obtained by the linear analysis adopting a reduced stiffness may give sufficient information for the verification of the limit state of deformation.

5.5.4. Linear analysis followed by limited redistribution of bending moments

For built-in continuous slabs, the numerically largest moments resulting from a linear analysis may be reduced without verifying the rotation capacity, provided the modification factor δ of the moments satisfies the same conditions given in clause 5.4.3.2 for beams and frames.

The reduction of the moment shall be performed assuming the average value for an appropriate width, providing that the average moments for the same width at the corresponding section (supports or midspan) are adjusted to satisfy equilibrium.

The redistribution may modify the moments at the same places in the other direction.

5.5.5. Plastic analysis

As this analysis is aimed for the ultimate behaviour, additional boundary conditions regarding rotation capacity and serviceability have to be satisfied. They are

The method should not be employed if type B steel is used or when considerations of second order effects or fatigue are required.

(a) The tensile reinforcement at any point and in any direction should not exceed one half of that which corresponds to a section for which the ULS in bending is characterized by the following strains

$$\varepsilon_s = \varepsilon_y \text{ and } \varepsilon_c = -0.0035 \; (\varepsilon_y = f_y/E_s)$$

In any case, the minimum amount may be disposed as for beams.

For built-in or continuous slabs the ratio of the support moments to the mid-span moments should normally not be less than 0.5 or more than 2.

This condition is required to avoid excessive cracking.

(b) For ribbed slabs it may be necessary also to verify the shearing force capacity of the reinforcement and the compression zone considering the crack depth.

5.5.5.1. Lower bound solution of plastic analysis

Statically admissible moment fields which satisfy the equilibrium condition may be found directly (e.g. by applying the strip method) or by starting from a linear analysis.

With this approach, even with the above mentioned limitations, the satisfaction of the serviceability limit states is rather uncertain. Therefore, it is convenient to adopt a field of moments which does not differ substantially from the elastic solution.

5.5.5.2. Upper bound solutions of plastic anlaysis

Kinematically admissible plastic deformation modes (e.g. the yield line theory) may be applied covering the corresponding moment field by securing the design ultimate moments at the cross-sections along the necessary yield lines.

5.5.6. Non-linear analysis

Application of the physically non-linear theory includes the interaction of all three moments in a slab, m_x, m_y, m_{xy}, and analyses the elastic non-cracked, cracked, and plastic phases of a slab subjected to increasing loads. The non-linear anlaysis covers the serviceability as well as the ultimate limit state.

For the numerical evaluation layered models, finite elements, or finite differences may be chosen.

5.6. DEEP BEAMS AND WALLS

5.6.1. Methods of analysis

The forces acting in the middle plane of a deep beam may be determined by applying either

(a) linear analysis based on the theory of elasticity
(b) statically admissible stress fields, in accordance with the lower bound theorem of limit analysis (as by analysing an equivalent truss consisting of struts and ties, preferably following the elastic field)
(c) non-linear analysis.

5.6.2. Linear analysis

The theory of elasticity may be applied assuming values of Poisson's ratio of 0.0 to 0.2. In most cases only numerical solutions (e.g. finite differences, finite element methods, or boundary element methods) are suitable. The analysis gives the fields of principal stresses and deformations. High stress concentrations, like those at the corner of openings, may be reduced considering cracking effects.

The linear analysis is valid both for serviceability and ultimate limit states.

The analysis for the ULS requires a correct detailing of the reinforcement to withstand the resultant tensile zones in the concrete, satisfying equilibrium conditions.

5.6.3. Analysis by statically admissible stress fields

If a stress field is chosen which satisfies the equilibrium conditions, a lower bound solution of limit analysis is considered. For the structure and its loads an equivalent truss may be investigated, consisting of concrete struts and arches as compressive members, and of steel ties, formed by the reinforcement as tensile elements, and their connections (nodes). The equilibrium model may be applied for verifying the ULS and also for the SLS, provided that the evaluated stress distribution is close to the results of the linear analysis.

This method is not allowed if type B steel is used.

5.6.4. Non-linear analysis

For a more refined analysis, non-linear stress–strain relations may be taken into account by applying the numerical method for two-dimensional plane

structures. The analysis then gives results for the serviceability as well as for the ultimate limit states.

5.7. SHELLS AND FOLDED PLATES

For two-dimensional structural elements subjected to combined bending and membrane action, usually a linear analysis of SLS, on the basis of the linear theory of elasticity, may suffice to evaluate the stress fields and deformations of the structure. The analysis should give principal forces and moments.

Shells subjected to compression have to be investigated with regard to buckling failure.

A solution satisfying equilibrium conditions may be found by considering only the membrane solution and neglecting the bending moments. The structure, however, may crack considerably especially at boundary regions (e.g. clamped edges). Hence, in general, bending should be included in the analysis.

At least the following effects should be included in such an analysis: geometrical imperfections of the shell form, the decrease of the bending stiffness caused by bending cracks (state II), displacements due to load actions and creep and settlement of supporting members at the boundaries.

5.8. STRUCTURAL EFFECTS OF TIME-DEPENDENT PROPERTIES OF CONCRETE

5.8.1. General

The inelastic strains due to creep and shrinkage of concrete may cause non-negligible changes in the long-term state of deformation and/or stress of structures and structural elements.

The characteristics and time sequence of loading and restraint conditions in the construction and service stages are of importance.

By analogy with the effects of other inelastic strains, the performance with respect to serviceability is primarily concerned.

Influence on the safety margins against collapse depends on the ductile behaviour of the structure or of the element and may become significant in cases where collapse is governed by non-plastic failure of concrete.

In slender or thin structures and whenever second order effects are of importance, the increase of deflections due to creep reduces the long-term safety margin with respect to buckling instability and may lead to creep buckling.

The influence of creep on second order effects is dealt with in a simplified way in clause 6.6.3.3.2. For complex problems and non-linear creep buckling reference should be made to specialized literature.

When choosing the level of refinement for the analysis of creep and shrinkage effects the following aspects should be considered

- reliability of the information on material properties (e.g. simple prediction on the basis of prediction laws of the type given in subsection 2.1.6, or prediction accompanied by test control at early ages, or test extrapolation; mean cross-section behaviour or local material properties etc.)

As evidenced in subsection 2.1.6, the available information on the time-dependent behaviour of concrete may suffer from important sources of error due to both lacunae in the model and to randomness in material parameters. Ambient conditions may be subjected to important variations whose influence on the real structure may be difficult to model in the

analysis. The attention given to the analysis should therefore be related in particular to the accuracy in establishment of the material parameters, avoiding an excessive refinement in the analysis if the prediction of material parameters is poor.

- importance of the effects of creep and shrinkage on the behaviour of the structure
- importance of the limit state under consideration.

For a complete review of methods of structural analysis for creep and shrinkage effects refer to the CEB Design Manual 'Structural Effects of Time-dependent Behaviour of Concrete', CEB Bulletin No. 142/142 *bis*, Lausanne, 1984, Georgi Publ. Co., St. Saphorin (CH), 1984.

This section applies essentially to the verifications with respect to serviceability limit states.

Creep and shrinkage properties are described in clause 2.1.6.4 as mean cross-section properties; therefore, their use for the determination of internal stresses and strains in time within cross-sections introduces larger errors. A realistic analysis of these effects by appropriate discretization techniques, should in principle be based on the description of local rheological properties, taking into account their intrinsic non-linearities, coupling with moisture and temperature distributions and non-linear effects of cracking.

5.8.2. Structural models

The overall analysis of structures for creep effects in terms of sectional forces and displacements may be conducted under the assumption of linearity, and of the consequent validity of the principle of superposition, as indicated in clause 2.1.6.4 for service stress levels $|\sigma_c| < 0.4 f_{cm}(t_o)$ (linear ageing viscoelastic model).

Non-linearities due to higher stress levels may be taken into account on the basis of clause 2.1.6.4.3d.

5.8.3. Application of linear model

5.8.3.1. General

In the linear ageing viscoelastic model the time-dependent behaviour of concrete is fully characterized by the creep function $J(t, t_0)$ as indicated in clause 2.1.6.4 for both constant and variable stress histories (eqs (2.1–62, 63)).

Alternatively with the same assumptions and range of validity the stress response to a variable imposed strain history may be written as

$$\sigma_c(t) = [\varepsilon_c(t_0) - \varepsilon_{cn}(t_0)]R(t, t_0) + \int_{t_0}^{t} R(t, \tau)d[\varepsilon_c(\tau) - \varepsilon_{cn}(\tau)] \quad (5.8\text{–}1)$$

where $R(t, t_0)$ is the relaxation function, representing the stress response to a constant unit imposed stress-dependent strain

$$[\varepsilon_c - \varepsilon_{cn}] = \varepsilon_{c\sigma} = 1$$

In the analysis of the global time-dependent behaviour of reinforced or prestressed concrete structures with rigid restraints, in terms of internal forces and displacements, the heterogeneities due to the presence of the

For practical application of linear creep analysis, a distinction between homogeneous structures with rigid restraints and heterogeneous structures is convenient.

steel, as well as to limited variations in the concrete properties between different parts, may be neglected in many cases and the structures be considered as homogeneous.

One should distinguish also between structures under constant restraint conditions and structures subjected to modifications of restraint conditions (variations of statical system) at some stage of construction and/or lifetime of the structure.

Application of the linear model to typical structural problems, in particular for homogeneous structures with rigid restraints, is facilitated, if both the creep function J and the corresponding relaxation function R are available (see clause 5.8.3.2).

The relaxation function R is obtained from the specified creep function J through the integral eq. (2.1–63) for $(\varepsilon_c - \varepsilon_{cn}) = \varepsilon_{c\sigma} = 1$.

Structural problems referring to heterogeneous structures are governed by one or more integral equations (see clause 5.8.3.3).

One of the alternative approaches of subsection 5.8.4 shall be used for the solution of integral equations in practical structural problems.

5.8.3.2. Homogeneous concrete structures with rigid restraints

Constancy of creep. Poisson ratio v shall be introduced as an additional assumption.

The analysis may be performed on the basis of the theory of linear visco-elasticity.

The stresses $\sigma_{ij}(t)$ and displacements $u_i(t)$ are then obtained on the basis of the stresses $\sigma_{ij,el}(t)$ and displacements $u_{i,el}(t)$ for an elastic structure of constant modulus.

For a reference modulus $E = E_c$ (clause 2.1.4.2) the following criteria shall be applied for the specified imposed actions or restraint conditions.

An addendum to CEB Bulletin 142/142 *bis* (to be published as CEB Bulletin 215) contains the graphs of the adimensional creep and relaxation functions $J(t, t_0)E_c$ and $R(t, t_0)/E_c$ corresponding to the creep function J specified in clause 2.1.6.4.3.

(a) Imposed loads

The elastic stresses are not modified by creep (i.e. $\sigma_{ij}(t) = \sigma_{ij,el}(t)$), while the displacements $u_i(t)$ may be obtained by integrating in time the increments of the elastic displacements $du_{i,el}(\tau)$, multiplied by the creep factor $J(t, \tau)E_c$ (first theorem of linear viscoelasticity).

(b) Imposed deformations

The elastic displacements are not modified by creep (i.e. $u_i(t) = u_{i,el}(t)$), while the stresses $\sigma_{ij}(t)$ may be obtained by integrating in time the increments of the elastic stresses $d\sigma_{ij,el}(\tau)$, multiplied by the relaxation factor $R(t, \tau)/E_c$ (second theorem of linear viscoelasticity).

(*c*) *Modification of the restraint conditions after the application of loads*
A modification of the restraint conditions, by the introduction of additional restraints at time $t = t_1$, following the application at $t = t_0$ of a system of constant sustained loads, produces a time-dependent variation of the initial elastic stresses and restraint reactions, which cannot be neglected generally.

The long-term distribution of stresses and reactions in a structure whose restraint conditions are modified shortly after loading (when the concrete is still young), may be considered to approach the elastic distribution corresponding to an application of loads to the final statical system of the structure.

Modification of restraint conditions after loading is frequent in practice; if the material behaviour is time-dependent, as for concrete structures, the long-term stress distribution may be widely affected, depending on age and creep deformability of the concrete.

5.8.3.3. Heterogeneous structures

The use of the linear ageing viscoelastic model for the concrete portion and of the elastic model for the steel elements leads to compatibility equations in terms of integral equations.

The procedures presented in subsection 5.8.4 can be applied. Attention should be paid, however, to the selection of the proper approach to obtain adequate accuracy for the different types of problems. Some specific procedures have been developed in the literature when the heterogeneity is due only to the presence of steel.

Heterogeneities may be due either to the properties of concrete (e.g. differences in casting ages, mix, size of structural elements, etc.), or to the presence of steel. They may characterize the cross-sections, and/or be diffused throughout the structure, and/or be concentrated in some associated structural element or external restraint.

When the analysis concerns stresses and strains within cross-sections, reference to the comments of subsection 5.8.2 is appropriate in selecting procedures and evaluating results.

(*a*) *Concrete structures made with different concrete portions*
Caution is needed in the application of the algebraic method of clause 5.8.4.3, as the different ages and rheological properties of concrete portions have to be considered; numerical solutions are therefore generally to be preferred.

If the analysis concerns heterogeneous sections, compatibility equations may be written under the usual assumption of sections remaining plane.

(*b*) *Heterogeneous steel–concrete sections*
Compatibility equations may be written under the assumption of sections remaining plane. A system of two integral equations is obtained by the displacement method if the steel cross-section has a non-negligible inertia, and a single equation if it may be reduced to a single fibre.

The use of the algebraic method of clause 5.8.4.3 normally leads to adequate accuracy.

The error introduced by adopting for the concrete a creep model representing the mean cross-section behaviour is diminished if the importance of the concrete portion in comparison with the steel is reduced (e.g. usual type of composite steel–concrete beam sections).

(*c*) *Concrete structures with external elastic restraints*
For structures with n elastic restraints the solution by the force method leads to a system of n integral equations.
The use of the algebraic method of clause 5.8.4.3 normally leads to adequate accuracy.

Forcing the elastic restraints (e.g. prestressing of steel cables in cable-stayed concrete bridges) up to the values of the reactions corresponding to rigid restraint conditions, eliminates the influence of creep deformability on the long-term stress regime, according to the first theorem of linear viscoelasticity (see clause 5.8.3.2a).

5.8.4. Practical approaches

The following practical approaches, with different degrees of refinement, may be adopted in structural calculations to deal with the integral equations due to the use of the linear ageing viscoelastic model

- numerical solutions
- conversion to differential forms
- conversion to algebraic expressions
- approximate determination of the relaxation function.

Because of the errors in creep prediction and in the linear model itself, the gain in accuracy achieved by the most refined linear methods, e.g. numerical solutions, is often fictitious compared to the use of the less refined approaches.

Numerical solutions are frequently used as reference procedures to evaluate the accuracy of approximate solutions.

5.8.4.1. Numerical solutions

Numerical step-by-step solutions may be obtained applying the trapezoidal rule to replace the superposition integral in the equations.
The relaxation function R corresponding to the specified creep function J is obtained according to clause 5.8.3.1 by applying the same procedure to the solution of eq. (2.1–63).

Proper subdivision of time in steps allows any desired accuracy; for usual time histories of the specified variable, it is convenient to use increasing steps.
For computer programs refer to CEB Bulletin 142/142 *bis*.

5.8.4.2. Conversion to differential forms

Rate-type stress–strain differential relations instead of integral-type relations may be obtained approximating, with any desired accuracy, the creep function by rate-type laws based on chains of rheological models with age-dependent parameters.

This procedure is recommended for complex problems involving a large number of time-dependent unknown variables, e.g. in finite-element visco-elastic analysis.

5.8.4.3. Approximate algebraic expressions

Adequate accuracy is obtained in most cases by converting the superposition integral in the equations into simplified approximate algebraic expressions and performing calculations in one time-step.

The following expression (age-adjusted-effective-modulus method), may be used instead of eq. (2.1–63) for a creep function J of the type of eq. (2.1–62)

$$\varepsilon_c(t) = \sigma_c(t_0)J(t, t_0) + [\sigma_c(t) - \sigma_c(t_0)]\left[\frac{1}{E_c(t_0)} + \chi(t, t_0)\frac{\phi(t, t_0)}{E_c}\right] + \varepsilon_{cn}(t)$$

$$= \frac{\sigma_c(t_0)}{E_{c,ef}(t, t_0)} + \frac{\sigma_c(t) - \sigma_c(t_0)}{E_{c,adj}(t, t_0)} + \varepsilon_{cn}(t) \qquad (5.8\text{–}2)$$

having introduced the ageing coefficient

$$\chi(t, t_0) = \frac{1}{1 - R(t, t_0)/E_c(t_0)} - \frac{1}{E_c(t_0)J(t, t_0) - 1}$$

$$= \frac{E_c(t_0)}{E_c(t_0) - R(t, t_0)} - \frac{E_c}{E_c(t_0)\phi(t, t_0)} \qquad (5.8\text{–}3)$$

the effective modulus

$$E_{c,ef}(t, t_0) = \frac{1}{J(t, t_0)} = \frac{E_c(t_0)}{1 + (E_c(t_0)/E_c)\phi(t, t_0)} \qquad (5.8\text{–}4)$$

and the age-adjusted effective modulus

$$E_{c,adj}(t, t_0) = \frac{E_c(t_0)}{1 + \chi(t, t_0)(E_c(t_0)/E_c)\phi(t, t_0)} \qquad (5.8\text{–}5)$$

5.8.4.4. Approximate determination of the relaxation function

Accurate values for R may be obtained by the semi-empirical expression

$$R(t, t_0) = \frac{1 - 0.008}{J(t, t_0)} - \frac{0.115}{J(t, t-1)}\left[\frac{J(t - \Delta, t_0)}{J(t, t_0 + \Delta)} - 1\right] \qquad (5.8\text{–}7)$$

with $\Delta = (t - t_0)/2$.

Eq. (5.8–2), with the expression (5.8–3) for the ageing coefficient χ, corresponds exactly to eq. (2.1–53) for all problems resulting from linear combinations of a creep and a relaxation problem. With sufficient accuracy its use may be extended to cover a large number of other asymptotic stress and strain time histories representing a variety of typical problems in creep analysis of structures.

An addendum to CEB Bulletin 142/142 *bis* (to be published as CEB Bulletin 215) contains the graphs of the ageing coefficient $\chi(t, t_0)$ corresponding to the creep function J specified in clause 2.1.6.4.3. Its values vary mainly between 0.5 and 1.0. In a large number of practical cases, particularly for average creep values and long-term creep effects, sufficiently accurate results may be obtained taking $\chi = t_0^{0.5}/(1 + t_0^{0.5})$; a constant value $\chi = 0.8$ can be adopted for typical loading ages between 10 and 30 days.

Adopting a constant value of $\chi = 1$, the following less accurate simple expression may be used (effective modulus method)

$$\varepsilon_c(t) = \sigma_c(t)J(t, t_0) + \varepsilon_{cn}(t) = \frac{\sigma_c(t)}{E_{c,ef}(t, t_0)} + \varepsilon_{cn}(t) \qquad (5.8\text{–}6)$$

Eq. (5.8–6) reduces to the calculation (with the effective modulus $E_{c,ef}$) of an equivalent elastic strain for the final value of the stress $\sigma_c(t)$ at the end of the time interval $(t - t_0)$ considered. Application of this method gives acceptable accuracy only for those cases where stresses are almost constant in time (varying less than 10% to 20%).

Eq. (5.8–7) may be used for any given creep function J; maximum error normally does not exceed 10% of the initial value of R.

Alternatively, the following expression obtained from eq. (5.8–3) offers similar accuracy, when the approximate values of χ indicated in clause 5.8.4.3 are introduced

$$R(t, t_0) = E_c(t_0)\left\{1 - \frac{\phi(t, t_0)}{[E_c/E_c(t_0)] + \chi\phi(t, t_0)}\right\} \qquad (5.8\text{–}8)$$

Negative values of R obtained from eqs (5.8–7) and (5.8–8) in extreme cases (e.g. low values of t_0 and of the relative humidity RH) should be discarded taking for R a positive value approaching zero.

6. VERIFICATION OF THE ULTIMATE LIMIT STATES

6.1. GENERAL APPROACH

6.1.1. Introduction

This chapter gives methods of verifying that, for a structure as a whole and for its component parts, the probability of an ultimate limit state being reached by the exceedance of the resistance of critical regions is acceptably small.

The ultimate limit state of static equilibrium is treated in chapter 1.

The determination of the action effects should be carried out in accordance with subsection 5.3.2.

Prestressing may be taken into account by treating the prestress as a loading system. The prestress loading system may be taken to incorporate a part of the other external loading balanced by the effects of the prestress. The magnitudes of the forces in this system should usually be calculated for prestressing forces $P_{do}(x, t)$, i.e. design values of the prestress corresponding to zero stress in the surrounding concrete.

Other treatments of prestressing are not excluded.

This definition of $P_{do}(x, t)$ assumes recovery of elastic losses of prestress. An exception to its use is noted in clause 6.4.2.2.

The parts of the resistance of the tendons not used in realizing these forces may be taken into account in the functions determining the resistance of the member to loads applied after the prestressing and bonding of the tendons.

In circumstances where the results of a verification are especially dependent on the magnitudes of prestressing forces a more refined treatment of them involving consideration of characteristic forces may be justified and reference should be made to clauses 1.4.3.2 and 1.4.3.3.

The determination of resistance should be based on physical models of the internal forces and external reactions of the structure.

The models should represent continuous systems of internal forces in equilibrium under the design ultimate conditions and should at least approximately consider compatibility of deformations.

6.1.2. Members with only bonded reinforcement

In the case of linear members, where the output of the analysis is values of M, N, V, T the verification should usually be made according to section 6.3. However, for hollow and flanged sections where transverse bending of the section and/or the combined effects of torsion and longitudinal shear are considered reference should be made to section 6.5.

For slabs with the output of analysis in terms of m_x, m_y, m_{xy} and transverse shear the verification should follow section 6.4.

For deep beams and other plates subjected to in-plane loading, if the output of the analysis is in terms of n_x, n_y and n_{xy}, the verification should be in accordance with subsection 6.5.3. If the analysis is a lower bound

solution (limit analysis) as in subsection 5.6.3 the models of section 6.8 or analogous models can be adopted.

For plates or slabs subjected to both in-plane and out-of-plane loading the verification should follow subsection 6.5.4.

Verification of local conditions at statical or geometric discontinuities should be in accordance with section 6.8.

6.1.3. Members with unbonded reinforcement

Unless an appropriate analysis of the whole system is carried out, the unbonded reinforcement (generally prestressing tendons) may be considered as separate elements acting on the reinforced concrete member. The forces exerted on the member should be given their initial design values reduced by the effects of creep and shrinkage, unless their variations are assessed by means of an appropriate analysis, which should take into account the displacements of the anchorages and of the deviators, as well as relevant second order effects (if any).

6.1.4. Combination of stress fields

Stress fields may be superimposed upon one another but the superimposition should avoid the addition of tensions and compressions at small angles to one another.

In general it is preferable that a single model should be derived for each ULS verification but the use of superimposition may be convenient in some cases.

As a guide an angle of 15° can be treated as a minimum for the separation of tensions and compressions in superimposed models.

Where two uniaxial compression stress fields are superimposed the design should respect the upper limit on the maximum compression resulting from the two.

$$\sigma_{max} = \sigma_1 \cos^2 \theta + \sigma_2 \cos^2 (\alpha - \theta) \qquad (6.1\text{–}1)$$

where

σ_1, σ_2 are greater and lesser uniaxial stresses
α is the angle between σ_1 and σ_2 ($\alpha < 90°$)
θ is the angle between σ_{max} and σ_1
$\cot 2\theta = (\sigma_1 + \sigma_2 \cos 2\alpha)/(\sigma_2 \sin 2\alpha)$,

and compressive stresses are positive.

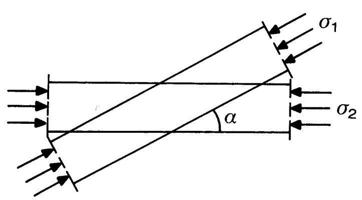


Fig. 6.1.1. Superimposed compression fields

6.1.5. Reinforcement and anchorage

The development of the forces required in the reinforcement and evaluated according to one of the available models of internal behaviour has to be ensured by bond and/or end anchorages. Consequently adequately anchored reinforcement shall be provided over the entire area in tension, across the trajectories of the compressive forces, at the locations where the directions of the aforementioned trajectories are deviated.

The axes of the ties in the model should coincide with the axes passing through the centroids of the reinforcement.

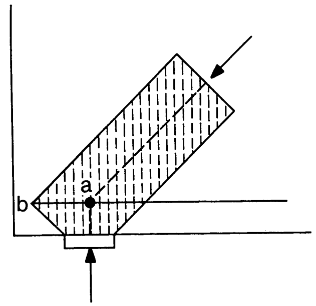


Fig. 6.1.2. Reinforcement and anchorage

The point 'a' is the node at which the centre-lines of compression and tension meet. The shaded areas denote zones of concrete in compression. Reinforcement is required out at least to the point 'b'.

Furthermore, the bar must be anchored in accordance with subsection 6.9.2.

Criteria for verification of resistance to fatigue are given in section 6.7.

6.2. MATERIAL RESISTANCES

6.2.1. General

The resistances given in the following subsections correspond to an increase of load from the service value to the design ultimate value over a period of hours or days. If the rate of loading is significantly greater (impact loading), these values may be replaced by ones appropriate to the rate in question.

Information on the effect of loading rates is given in section 2.1.

Note that in this chapter the sign-convention for stresses and strains differs from the one used in section 2.1.

6.2.2. Concrete in compression

6.2.2.1. General

It should be verified that, under the relevant ULS conditions, the maximum compressive force acting on an area of concrete does not exceed a limit value, corresponding to the resultant of the resisting stresses, as given by the constitutive laws of section 2.1 and the appropriate safety factors.

However, appropriate simplifications of these constitutive laws (see clause 2.1.4.4) are allowed especially for cases where the zones checked are subjected to essentially uniaxial compression.

The expression 'essentially uniaxial compression' is used here to describe the stress state in the struts of models, e.g. in the compression chords of beams subjected to shear and bending, which are modelled as simple struts even though a more detailed analysis would indicate the presence of biaxial tension-compression.

6.2.2.2. Essentially uniaxial compression

Two alternative simplifications of the basic constitutive laws (see clause 2.1.4.4) which are appropriate are

— a parabola-rectangle stress–strain diagram,
— a uniform stress diagram.

Parabola-rectangle diagram

The design resistance of an uncracked zone under essentially uniaxial compression may be determined by means of a parabola-rectangle diagram as follows

$$\left.\begin{aligned} \sigma_{cd} &= 0.85 f_{cd}\left[2\left(\frac{\varepsilon_c}{\varepsilon_{c1}}\right) - \left(\frac{\varepsilon_c}{\varepsilon_{c1}}\right)^2\right] && \text{for } \varepsilon_c < \varepsilon_{c1} \\ \sigma_{cd} &= 0.85 f_{cd} && \text{for } \varepsilon_{c1} \leqslant \varepsilon_c \leqslant \varepsilon_{cu} \\ \sigma_{cd} &= 0.00 && \text{for } \varepsilon_{cu} < \varepsilon_c \end{aligned}\right\} \quad (6.2\text{–}1)$$

The coefficient 0.85 and the use of a constant stress for strains from ε_{c1} to ε_{c2} allow for the influence of long-term loading.

where $\varepsilon_{c1} = 0.002$.

For flexure

$$\left.\begin{aligned} \varepsilon_{cu} &= 0.0035 && \text{for } f_{ck} \leqslant 50\,\text{MPa} \\ \varepsilon_{cu} &= 0.0035\left(\frac{50}{f_{ck}}\right) && \text{for } 50\,\text{MPa} < f_{ck} \leqslant 80\,\text{MPa} \end{aligned}\right\} \quad (6.2\text{–}2)$$

For axial compression

$$\varepsilon_{cu} = 0.002 \quad (6.2\text{–}3)$$

The strains ε_c are absolute values and are positive for compression.

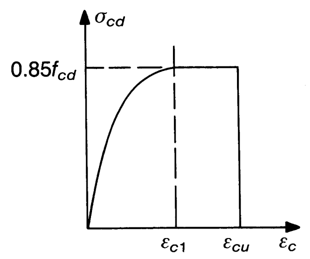


Fig. 6.2.1. Parabola-rectangle stress–strain diagram

Uniform stress diagram

The design resistance of a zone under essentially uniaxial compression may also be determined by means of a further simplified uniform stress diagram over the full area of this zone if appropriately selected.

The average stress may be taken as

$$f_{cd1} = 0.85\left[1 - \frac{f_{ck}}{250}\right] f_{cd} \quad (6.2\text{–}4)$$

for uncracked zones, or as

$$f_{cd2} = 0.60\left[1 - \frac{f_{ck}}{250}\right] f_{cd} \quad (6.2\text{–}5)$$

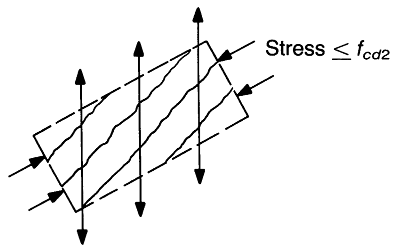


Fig. 6.2.2. Example of reduced resistance f_{cd2}

for cracked zones where the compressive resistance may be reduced by the effect of transverse tension from reinforcement and by the need to transmit force across the cracks.

These values are valid under the condition that the maximum extreme fibre strain is taken as

$$\varepsilon^*_{cu} = 0.004 - 0.002 \frac{f_{ck}}{100} \quad (6.2\text{–}6)$$

f_{ck} is in MPa.

6.2.2.3. Biaxial and triaxial stress states

For biaxial and triaxial stress states reference is made to sections 2.1 and 3.4.

6.2.2.4. Compression fields affected by bars or ducts

If a compression zone contains bars or ducts, such that the sum of the diameters of the bars or ducts at one level is greater than 1/6th of the width of the zone, or the sum of their areas is greater than 4% of the area of the zone, account shall be taken of their effect on the compressive stresses.

No reduction of breadth is required for ordinary bonded bars at the boundary of the compressed zone, e.g. for ordinary main steel at the boundary of the web of a beam.

Compressive stresses inclined to bars or ducts shall be calculated on the basis of a reduced breadth

$$b_{red} = b - \eta\Sigma\phi \quad (6.2\text{–}7)$$

where

This section applies particularly to the webs of prestressed concrete beams, in which case $b = b_w$.

b is the breadth of the compression zone

$\Sigma\phi$ is the sum of the diameters of the bars or ducts determined at the most unfavourable level

η is a coefficient depending on the nature of the bars or ducts.

Indicative values for η are

$\eta = 0.5$ for bonded bars or grouted tendon ducts,

$\eta = 1.2$ for unbonded tendons and ungrouted ducts.

The value $\eta = 1.2$ is intended for simple cases of individual tendons or ducts if no extra reinforcement is provided for the transverse tension around the embedment. If suitable transverse reinforcement is provided $\eta = 1.0$.

Due consideration shall be given to the transverse tension produced by the local deviation of the compression field, and to the provision of transverse reinforcement.

Compressive stresses parallel or nearly parallel to ducts should be calculated assuming the area in compression to be reduced by the area of the ducts within it.

6.2.2.5. Compression zones with confinement

Account may be taken of the effect of appropriately closed hoops or helical reinforcement giving a triaxial behaviour to concrete under longitudinal compression.

This may be done using the stress–strain diagrams for confined concrete given in clause 3.5.2.1.

6.2.3. Concrete in tension

The tensile resistance of concrete should not normally be relied upon in any major tie and therefore no general criteria are given for the use of concrete tensile strength. Concrete is, however, relied upon to provide tensile resistance in association with bond/anchorage, shear in members without shear reinforcement, etc. Where such reliance is envisaged, explicit criteria are given.

6.2.4. Steel in tension

The design strength of steel in tension is as follows.

For ordinary reinforcement

$$f_{ytd} = f_{ytk}/\gamma_s \tag{6.2–8}$$

For bonded prestressed reinforcement, if the prestress is treated as an external load

$$f_{pyd,net} = 0.9 f_{ptk}/\gamma_s - \sigma_{do} \leqslant 600\,\text{MPa} \tag{6.2–9a}$$

where σ_{do} is the design tendon stress taken into account in the prestress loading system.

The attainment of the stress f_{ytd} or f_{pyd} requires the tensile strain to reach the design yield value. If this is not the case, the stress in the steel in the ULS may be derived from the strain and the stress–strain relationship for the reinforcement.

For the prestressed reinforcement, as an alternative to the use of eq. (6.2–9a) a factored stress–strain diagram derived from the actual characteristic relationship may be employed provided that the relevant ULS strains are calculated. In practice this is only generally possible for the sections of maximum moment in members free from torsion.

If prestress is not treated as an external load and the full tendon strength is to be used in a resistance function

$$f_{pyd} = 0.9 f_{ptk}/\gamma_s \leqslant \sigma_{do} + 600\,\text{MPa} \tag{6.2–9b}$$

6.2.5. Steel in compression

Bonded longitudinal reinforcement should be assumed to undergo the same changes of strain as the surrounding concrete. If the detailing requirements regarding lateral restraint to the bars (see clause 9.2.3.2) are satisfied the design strength of the steel in compression is

$$f_{ycd} = f_{yck}/\gamma_s \tag{6.2–10}$$

Only bars in the direction parallel to a compression strut should be considered effective.

The attainment of the stress f_{ycd} requires the compressive strain to reach the design yield value. If this is not the case, the stress in the steel at the ULS may be derived from the strain and the stress–strain diagram for the reinforcement.

Where bonded post-tensioned tendons are located in a zone which is in compression under the action of the applied loads for the ultimate limit state, the reduction of the prestressing force should not be taken to exceed that corresponding to a change of strain equal to 0.0015.

This limitation is intended to account for possible bond slip.

6.3. LINEAR MEMBERS

6.3.1. Basic assumptions

(*a*) For simple rectangular members subjected to axial load, flexure and shear, the design models consist of longitudinal chords connected by web lattices comprising concrete struts under diagonal compression and distributed reinforcement in one or more directions.

In the following it is assumed that the minimum thickness of concrete in any part of a member is 80 mm. For members with thinner parts special consideration should be given to dimensional tolerances and possible consequences for design.

For the case of moving loads or multiple load cases, it is necessary to consider the envelopes of the longitudinal forces using different models if relevant. Regarding stirrup forces and diagonal concrete compressive forces in reinforced concrete beams, shear force envelopes can directly be used.

(*b*) For more complex sections and loading conditions, the members can be subdivided into several wall elements (webs, flanges) which can then be designed for their individual action effects. Longitudinal shear in flanges and torsion of beams are treated in this way in subsections 6.3.4 and 6.3.5.

For a more generalized treatment of complex loading conditions, including for example the simultaneous effects of torsion, longitudinal shear and transverse bending reference should be made to section 6.5.

6.3.2. Axial action effects

The resistant sectional forces should be derived from the internal resistant stresses and forces in the concrete and reinforcement on the basis of the following assumptions:

(*a*) the distribution of longitudinal strain is linear over the depth of the section;
(*b*) tensile stresses of concrete are neglected;
(*c*) bonded reinforcement is subjected to the same variations in strain as the adjacent concrete;
(*d*) the total deformations of bonded prestressing tendons are calculated taking account of the preliminary elongation corresponding to the design value of the prestressing force in the reference state after losses.

Section 6.3.2 is intended to be applied in cases where loading is only by axial action effects and at sections of maximum bending moment in other cases.

An exception to (c) the reduction of strain in prestressed tendons in a compressed zone should not be taken to exceed 0.0015 (see subsection 6.2.5).

On the basis of these assumptions, if the parabola-rectangle stress–strain diagram for concrete (6.2.2.2) is used, the sectional forces may be derived on the basis illustrated by Fig. 6.3.1 in which it is to be assumed that the strain diagram will pass through either point A or point B.

A diagram passing through A corresponds to simple or compound bending, while one passing through B corresponds to simple compression or compound bending with the section entirely in compression.

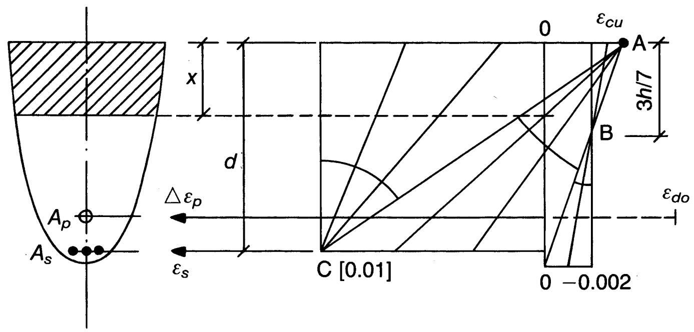


Fig. 6.3.1. Strain diagram (parabola-rectangle diagram): ε_{do} is the tendon strain corresponding to $P_{do}(x, t)$

In some cases, e.g. where different steel types are used or when several steel bars are distributed over the height of the section, it is suggested that $\varepsilon_{s,\max}$ or $\Delta\varepsilon_p$ is limited to 0.01. In this case the strain diagram in Fig. 6.3.1 will pass through point C instead of A or B.

In all cases

$$\varepsilon_{s,\max} \leqslant \varepsilon_{uk} \quad \text{or} \quad \varepsilon_{do} + \Delta\varepsilon_p < \varepsilon_{uk}$$

For ε_{uk} see clause 2.2.4.4 or 2.3.4.3.

If the uniform stress diagram is used for the concrete (clause 6.2.2.2), the maximum extreme fibre compressive strain is always limited to ε_{cu}^* and $\varepsilon_{s,\max}$ or $(\varepsilon_{pp} + \Delta\varepsilon_p)$ is limited as above.

For any combination of action effects the applied moment and axial force shall not exceed the corresponding resistances calculated on the basis of the assumptions given above and the criteria of section 6.2.

If prestress is treated as a loading system its effects should be included in the applied moment and axial force (see subsection 6.1.1).

An increase of compressive strength of concrete due to confinement may be taken into account according to clauses 6.2.2.5 and 3.5.2.1 provided that the area of the compression zone is taken as that of the confined concrete.

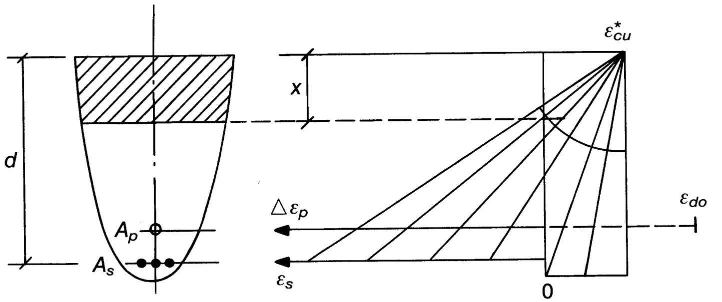


Fig. 6.3.2. Strain diagram (uniform stress diagram): ε_{do} is the tendon strain corresponding to $P_{do}(x, t)$

Beams are linear members mainly subjected to bending and shear without significant compression due to external actions.

Columns are linear members subjected to significant compression due to external actions. They are commonly also subject to significant bending and shear.

The range of models which might represent the detailed behaviour of linear members is very large on account of the range of possible cross-sections (which may be symmetrical or not, solid or not and have various proportions), of possible loading conditions (which may include also lateral bending and shear) and of the proportions between the components of the action effects.

It is assumed that the minimum measures related to longitudinal and transverse reinforcement are satisfied. However, linear members of minor importance such as lintels and floor joists interconnected so as to allow for redistribution of forces between joists may be designed without web reinforcement and should then comply with the requirements of section 6.4.

6.3.3. Shear and axial action effects

For the verification with regard to the ultimate limit state of resistance of critical regions three main models are presented below, respectively for

- reinforced concrete beams (clause 6.3.3.2)
- prestressed concrete beams (clause 6.3.3.3)
- reinforced concrete columns (clause 6.3.3.4).

6.3.3.1 Conditions for application of models

The application of the models given below is subject to the following conditions.

(*a*) The ratio of the tensile reinforcement should be limited (avoidance of over-reinforced sections) so that

$$0.0035 \frac{d - x}{d} > f_{yd}/E_s \qquad (6.3\text{–}1a)$$

or

$$0.0035\,\frac{d - x}{d} > f_{yk}/E_s\gamma_s \tag{6.3–1b}$$

which leads to a x/d-value approximately equal to 0.6.

(*b*) The mechanical ratio of stirrup reinforcement should be not less than 0.2, i.e.

$$\omega_{sw} = A_{sw}f_{yk}/(b_w s f_{ctm} \sin\alpha) \geqslant 0.2 \tag{6.3–2}$$

where s is the spacing of the stirrups (A_{sw}) measured along the axis of the member. For f_{ctm} see clause 2.1.3.3.

(*c*) The inclination of stirrups to the axis of the member should be at least 45° and that of bent-up bars at least 30°.

(*d*) The spacing of stirrup legs (in both the longitudinal and transverse directions) should not normally exceed the lesser of $0.75d$ and 800 mm.

(*e*) The shear reinforcement should be adequately anchored to the chords (see clause 9.2.2.2).

The fundamental 'unit-length' model of a typical part of the web of a beam resisting shear and axial action effects is shown in Fig. 6.3.3.

The angle θ between the web compression and the chords may be chosen freely in the range from 45° (arccot 1) to 18.4° (arccot 3).

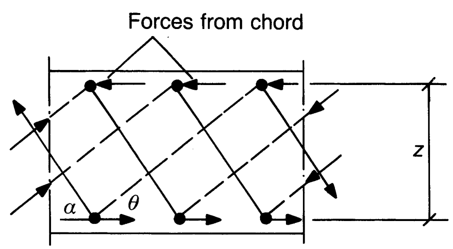


Fig. 6.3.3. Web model

The use of a high value for $\cot\theta$ increases the stresses in the shear reinforcement at stages between shear cracking and the ULS and also increases demands on the extension (anchorage) of the main steel. The crack control requirements of section 7.4 may then govern the design of the shear reinforcement especially in large members and may not permit $\cot\theta$ values as high as 3. The use of high values for $\cot\theta$ is not advised where a member is subjected to axial tension.

Longitudinal tension reinforcement should normally be contained within the stirrup cage.

The absolute maximum shear resistance for a given section and concrete strength is obtained with $\theta = 45°$.

$$V_{Rd,\max} = \frac{f_{cd2}}{2}\,b_w z(1 + \cot\alpha) \tag{6.3–3}$$

where f_{cd2} is given by eq. (6.2–5).

It may be noted that where inclined shear reinforcement is used greater values of $V_{Rd,\max}$ would theoretically result from the adoption of $\theta > 45°$. The realization of the greater resistance has, however, not yet been verified experimentally.

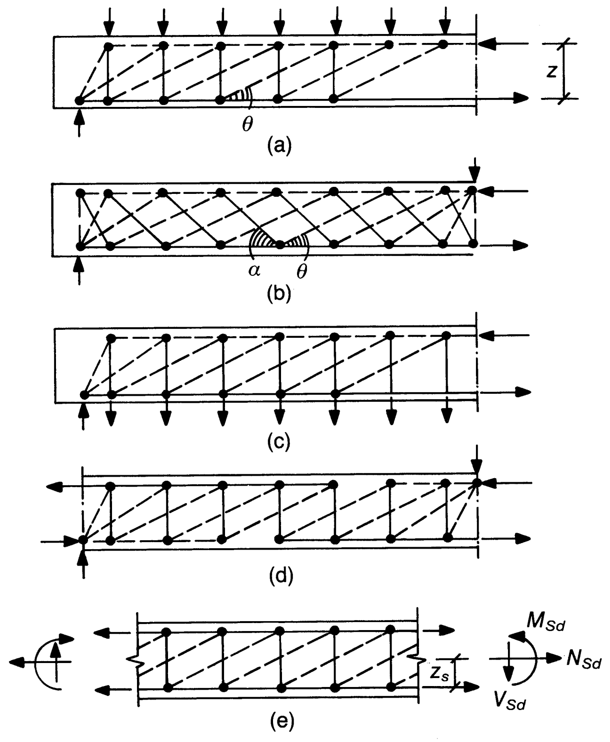


Fig. 6.3.4. Models for reinforced concrete beams with parallel chords: (a) continuous top load; (b) concentrated load; (c) continuous hanging load; (d) concentrated load, continuous beam; (e) region under bending, shear and axial tension*

6.3.3.2. Reinforced concrete beams

Chords parallel

Models for internal action in beams with parallel chords are given in Fig. 6.3.4. The values of the lever arm z and the depth x of the compression zone throughout a region in which bending moments retain the same sign may be taken equal to the values at the section of maximum M_{Sd}, evaluated according to chapter 5 on the basis of the assumptions introduced in subsection 6.3.2.

The forces derived from the models and the verifications required are as follows:

(*a*) *Tension chord*

Acting force

$$F_{St} = \frac{|M_{Sd}|}{z} + N_{Sd}\frac{(z - z_s)}{z} + \frac{V_{Sd}}{2}(\cot\theta - \cot\alpha) \qquad (6.3\text{–}4)$$

where N_{Sd} is the axial load, taken positive for tension and negative for compression.

In the case of support reactions/loads applied so as to create transverse compression over the depth of the member

$$F_{St} \leqslant \frac{|M_{Sd,\max}|}{z} + N_{Sd}\frac{(z - z_s)}{z} \qquad (6.3\text{–}5)$$

where z_s is the distance from the line of action N_{Sd} to the centroid of the tension reinforcement.

In cases where all the tension reinforcement is within the breadth of the web

$$F_{Rt} = A_s f_{yd} \qquad (6.3\text{–}6)$$

In cases where some of the tension reinforcement is outside the breadth of the web the force to be resisted by the reinforcement is generally greater than the chord force (see subsection 6.3.4). It is however limited by eq. (6.3–5) in cases of direct support/loading.

*Concerning beams with inclined chords, see the last part of this section.

(*b*) *Compression chord*

$$F_{Sc} = \frac{|M_{Sd}|}{z} - N_{Sd}\frac{z_s}{z} - \frac{V_{Sd}}{2}(\cot\theta - \cot\alpha) \qquad (6.3\text{–}7)$$

except at the section of maximum moment where for direct loading

$$F_{Sc} = \frac{|M_{Sd,\max}|}{z} - N_{Sd}\frac{z_s}{z} \qquad (6.3\text{–}8)$$

$$F_{Rc} = f_{cd1}A_c + f_{ycd}A_{sc} \qquad (6.3\text{–}9)$$

where A_c denotes the cross-sectional area of the compression chord.

(*c*) *Compression of web concrete*

$$F_{Scw} = \frac{V_{Sd}}{\sin\theta}\left(\frac{\cot\theta}{\cot\theta + \cot\alpha}\right) \qquad (6.3\text{–}10)$$

$$F_{Rcw} = f_{cd2}b_w z\cos\theta \qquad (6.3\text{–}11)$$

(*d*) *Tension of web steel*

$$F_{Stw} = \frac{V_{Sd}}{\sin\alpha} \qquad (6.3\text{–}12)$$

$$F_{Rtw} = \left[\frac{A_{sw}f_{yd}}{s}\right] z(\cot\theta + \cot\alpha) \qquad (6.3\text{–}13)$$

Generally no specific verifications are required for F_{Sc} other than at the section of $M_{Sd,\max}$ but see subsection 6.3.4 for the design of compression flanges.

The resistance f_{ycd} is reliable only if the strain of the compression reinforcement, calculated according to subsection 6.3.2, is sufficiently high.

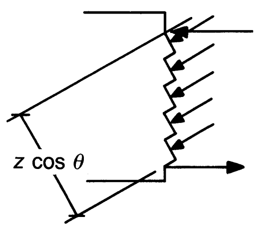


Fig. 6.3.5. Compression of web concrete

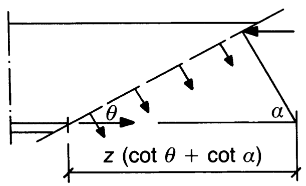


Fig. 6.3.6. Tension of web steel

In case of moving loads or multiple load cases, shear force envelopes may be directly used for dimensioning of stirrups provided that the angles α and θ are constant throughout.

For a region of a member in which the bending moment retains the same sign and the cross-section remains constant, the verification procedure may be as follows.

- *Stage I.* The section of maximum moment is designed according to subsection 6.3.2, unless conditions of indirect loading or support require the model shown in Fig. 6.3.7 in which case the resistance of the tension chord must be increased appropriately using eq. (6.3–4).
- *Stage II.* The resistance of the web concrete is verified at the section of maximum shear force.
- *Stage III.* The resistance of the web reinforcement is verified using eqs. (6.3–12) and (6.3–13)
  — for the region further from the support than $z \cot \theta$ in cases of direct loading (Fig. 6.3.8(a)) where the reinforcement determined $z \cot \theta$ from the support is continued to the support (but see stage V below for the cases where major loads act near supports),
  — for the entire region where load is not applied at the top of the beam (Fig. 6.3.7).

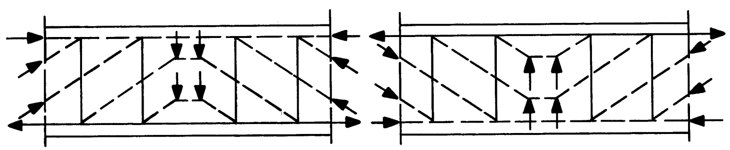


Fig. 6.3.7. Conditions at loads and supports distributed over the depth

- *Stage IV.* The admissibility of curtailing main reinforcement is determined by eqs (6.3–4) and (6.3–6).

  Higher values of $\cot \theta$ lead to lower amounts of stirrups but increase the forces in the main steel in regions of low moments.

  For the vertical stirrups alone, the minimum amount is obtained for

$$\cot \theta = \sqrt{\left[\frac{b_w s f_{cd2}}{A_{sw} f_{ywd}} - 1\right]} \leqslant 3 \qquad (6.3\text{–}14)$$

for which

$$\frac{V_{Sd}}{z b_w f_{cd2}} = \sqrt{\left[\frac{A_{sw} f_{ywd}}{b_w s f_{cd2}}\right]} \sqrt{\left[1 - \frac{A_{sw} f_{ywd}}{b_w s f_{cd2}}\right]} \leqslant 3 \frac{A_{sw} f_{ywd}}{b_w z f_{cd2}} \qquad (6.3\text{–}15)$$

- *Stage V*. In the case of a shear span in which a large part of the transverse loading is applied within a distance $z \cot \theta$ ($\leqslant 3z$) of a support, stage III above permits the shear reinforcement to be designed for only a small force. This has a number of consequences.
  - — The inclined compressive force at the support may be considerably increased and it should be verified that the compressive stresses at the nodes are not excessive — see subsection 6.9.2.
  - — The force in the main steel requiring anchorage at a simple support is increased and the adequacy of the anchorage should be verified — see subsection 6.9.3.
  - — If shear cracking would occur in the serviceability limit state the amount of shear reinforcement controlling the opening of the diagonal crack may be very small and the criteria of serviceability may be violated. In the absence of a more precise calculation the shear force causing shear cracking may be estimated as

$$V_{cr} = 0.15(3d/a_v)^{1/3} \xi (100 \rho f_{ck})^{1/3} b_{red} d$$

where

a_v is the distance from major load to support
$\xi = 1 + \sqrt{(200/d)}$ with d in mm
ρ is the ratio of flexural tensile reinforcement $(A_s/b_w d)$ anchored at the support
b_{red} is the reduced web breadth
$(3d/a_v)^{1/3}$ is an empirical expression allowing for the influence of the transverse compression from the loads and support reaction.

If the above present difficulties for the design, conditions can be improved by increasing the angle θ of the compression struts (Fig. 6.3.8(b)) or by sharing the load between a single direct thrust and a lattice system (Fig. 6.3.8(c)). In the latter case conditions in the strut at the support can be verified for the resultant of the two inclined thrusts.

Further guidance on the design of regions where major loads are applied close to supports is given in clause 6.8.2.2.

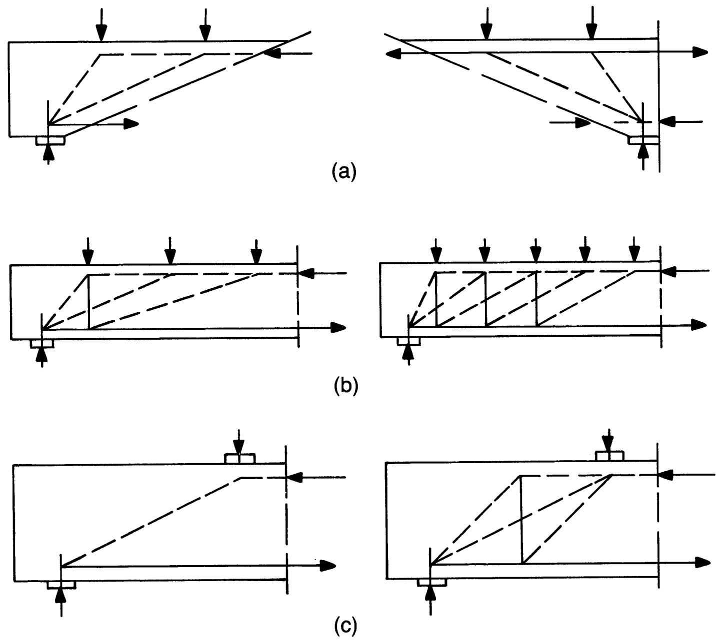


Fig. 6.3.8. Direct loading near supports: (a) support of loads close to supports by direct thrusts; (b) improvement of conditions at a support by increasing the angle θ; (c) improvement by sharing load between a direct thrust and a lattice system

An indicative model for a beam with both chords inclined is given in Fig. 6.3.9. The forces in the chords and web should be determined.

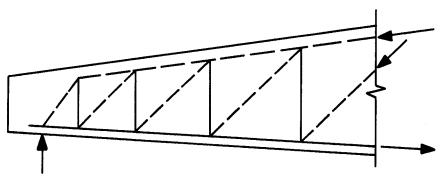


Fig. 6.3.9. Model for a beam with inclined chords

6.3.3.3. Prestressed concrete beams

6.3.3.3.1. General

Prestress is here treated as an external loading system in accordance with subsection 6.1.1.

Three modelling approaches are presented in the following three clauses and clause 6.3.3.3.5 treats special conditions, e.g. those at the ends of pretensioned members.

In all cases the verification is made in individual regions as defined in clause 6.3.3.2 or in zones as defined below. The verification criteria directly applicable are those relating to the 'first yield' defined in clauses 1.6.2.2 and 5.4.1.2.

In all three approaches, two models are superimposed.

The first model represents actions maintaining equilibrium with the prestress (end forces and forces due to tendon curvature) and a part of the other loading. This model shall be such that no reinforcement is necessary for the equilibrium. The model may include a compression arch with a force having a longitudinal component in equilibrium with the longitudinal component of the prestress (Fig. 6.3.10(a)). In this case the other loading considered is such as to maintain equilibrium with the transverse forces due to the curvature of the tendons and of the arch. This version of the first model, including the arching action can reduce the total amount of shear reinforcement required, especially in members with relatively thick webs.

An alternative version of the first model (Fig. 6.3.10(b)) does not include an arch and uses longitudinal forces in the chords to maintain equilibrium with the longitudinal effects of the prestress.

The second model is the truss analogy of an ordinary reinforced concrete beam (Fig. 6.3.10(c)). The forces on it complement those of the first model and are equivalent to the remainder of the actions for which reinforcement is required.

The first approach (*clause 6.3.3.3.2*) using a model of the type shown in Fig. 6.3.10(a) together with that of Fig. 6.3.10(c) is of general applicability.

The second approach (*clause 6.3.3.3.3*) is a special case of the first, in which the part of the loading applied to the first model is a constant (along the length) proportion of the total load. This approach simplifies the calculations required but is of restricted applicability. In the main it is applicable where the tendons are continuous and their profiles are such that

The models presented below are intended to be used for relatively simple cases of tendon layout, load conditions and section shapes. In other cases (box girder sections, high number of tendons, etc.), the decomposition of the section in plate elements (see section 6.5) may be more convenient in particular to operate on a finite element analysis output.

The choice of approach should depend on various considerations, e.g. the shape of the cross-section, the spatial arrangement of the external loads and of the prestressing tendons and the degree of refinement required.

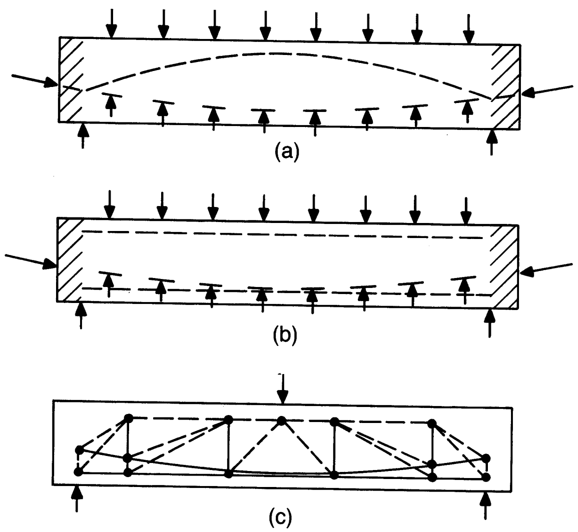


Fig. 6.3.10. Models 1 and 2 for prestressed beams: (a) model 1 with arch action; (b) model 1 without arch action; (c) model 2

The first model may in principle be a combination of the types illustrated in Figs 6.3.10(a) and (b).

the forces due to their curvature have a distribution similar to that of the loads, e.g. a simply supported beam with a parabolic tendon profile carrying uniformly distributed loads.

The third approach (*clause 6.3.3.3.4*) is the simplest and is of general applicability. The models used are those of Figs. 6.3.10(b) and (c). This approach is realistic for thin-webbed members and somewhat conservative in other cases.

6.3.3.3.2. General differentiated approach

In the case of internal prestress by post-tension, the beam is divided into zones (one zone only in the simplest case) generally defined by the points of contraflexure.

Model 1 consists of a compression arch (curved strut) totally internal to the beam. The centre-line of this arch should pass

The simplest case is that of a simple span having tendons without intermediate anchorages.

If several tendons are spread over the height of the section, the resultant force is considered to determine the points of contraflexure.

In cases where intermediate anchorages would exist within a zone, the model should be more differentiated (see clause 6.3.3.3.5).

In the case of pretension the tendons are generally straight and the limits between zones, if relevant, correspond approximately to zero moment points under the loading for the load case considered.

- through the centre of the compression chord at the section of maximum moment in the zone considered, and

The position of the centre of the compression chord of this section is defined by the design for axial action effects. In the normal case where the moment due to the other actions (loads) exceeds the moment due to prestress, the compression chord is that opposite the chord containing the tendons.

- through points at the extremities of the zone usually at the level of the tendons.

Other points may be adopted at the extremities provided an external moment transferred from model 2 completes the equilibrium of the end sections.

Within a given zone the prestressing force may be considered as constant and equal to the value $P_{do}(x, t)$ at the section of maximum moment defined in subsection 6.2.4.

The horizontal component of the force in the arch is taken as equal to the horizontal component of the prestress.

The profile of the arch between the points defined above may be chosen rather freely to suit the loading.

In the absence of concentrated loads at determined abscissae a parabolic or circular profile is recommended. If concentrated loads are applied at determined abscissae, the profile may advantageously include angles at these abscissae and be closer to straight lines between them.

The equilibrium of the arch must be ensured by the forces at its ends and forces distributed along it (due to the effects of tendon curvature and transverse loading) in such a way that the arch profile is a funicular of these forces.

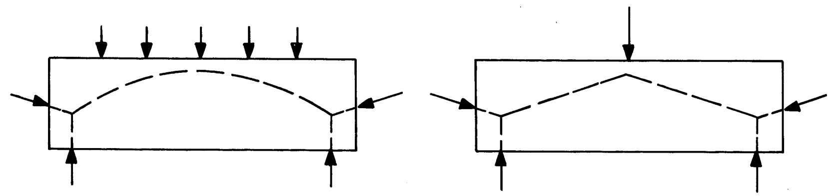


Fig. 6.3.11. Arch profiles adapted to different patterns of loading

Model 2 is the truss model of an ordinary reinforced concrete beam in which the remainder of the strength of the tendons not used in $P_{do}(x, t)$ is included, i.e. the design strength corresponding to $f_{pyd,net}$ as defined in subsection 6.2.4.

Model 2 is considered as subject to the following (design values)

- the part of the permanent actions (other than prestress) and the variable actions not applied to model 1
- end shear forces, statically determined, equilibrating the forces above
- hyperstatic end moments and shears of prestress
- end moments due to actions other than prestress, calculated in the overall analysis and equilibrating shears.

In the case where the extremities of the centre-line of the arch do not coincide with the level of the tendons a corrective moment shall be applied there.

The stress fields of the two models are superimposed for the final verifications.

The verifications required are of

- the force in the compression chord (sum of forces from models 1 and 2)
- the force in the tension chord (sum of forces from models 1 and 2, but note that the tension chord force is zero in Fig. 6.3.10(a))
- the web tension from model 2 (model 1 web tension is equal to zero)
- the web compression (sum of effects from models 1 and 2).

The simplified superposition approach of clause 6.3.3.3.3 may be adopted for the web compression.

6.3.3.3.3. Simplified differentiated approach

This approach is as above in clause 6.3.3.3.2 but the profiles of the arch and tendons are such that their combined curvature effects equilibrate a constant (along the length) fraction λ of the load.

The constant fraction refers to the external loading.

In this case the forces to which model 2 is subjected are simply the fraction $(1 - \lambda)$ of the permanent (prestress excluded) and variable loads together with eventual hyperstatic effects.

The forces derived from the models and the corresponding resistances are as follows

For sections other than that of maximum moment, reference should be made to the truss model.

(*a*) *Tension chord at the section of maximum moment*

$$F_{St} = (1 - \lambda)\frac{M_{Sd}}{z} - F_{pb} \qquad (6.3\text{–}16)$$

$$F_{Rt} = A_s f_{ytd} + A_p f_{pyd,net} \qquad (6.3\text{–}17)$$

For $f_{pyd,net}$ see subsection 6.2.4.

where F_{pb} is the compressive force applied to the tension chord in the model of Fig. 6.3.10(b).

(*b*) *Compression chord at the section of maximum moment*

$$F_{Sc} = (1 - \lambda)\frac{M_{Sd}}{z} + F_{c\lambda} \quad (6.3\text{–}18)$$

$$F_{Rc} = f_{cd1} A_c + f_{ycd} A_{sc} \quad (6.3\text{–}19)$$

where $F_{c\lambda}$ is the compressive force in model 1.

If prestressing tendons are present within the compression chord their influence should be treated in accordance with subsection 6.2.5.

(*c*) *Compression of web concrete*
Acting stress

$$\sigma_{Scw} = \sigma_1 + \sigma_2 \quad (6.3\text{–}20)$$

$$\sigma_1 \cong \frac{F_{c\lambda}}{b_w z}\sqrt{\left[\frac{c_v^2 + z^2}{c_v^2}\right]} \quad (6.3\text{–}21)$$

where c_v is the horizontal length between the section of maximum moment and the relevant end of the zone, and b_w complies with clause 6.2.2.4

$$\sigma_2 = \frac{(1 - \lambda)V_{Sd}}{b_w z \sin^2\theta(\cot\theta + \cot\alpha)} \quad (6.3\text{–}22)$$

The direct addition of σ_1 and σ_2 is in principle conservative and is adopted here partly for simplicity and partly because of the approximate nature of the expression for σ_1.

In practice with diffusion of compressive forces, this is a gross simplification to give the average compressive stress. z may be taken as 0.9 h.

For vertical stirrups

$$\sigma_2 = \frac{(1 - \lambda)V_{Sd}}{b_w z \sin\theta\cos\theta}$$

Resistant stress

$$\sigma_{Rd} = f_{cd2} \quad (6.3\text{–}23)$$

(*d*) *Tension of web steel*

$$F_{Stw} = (1 - \lambda)\frac{V_{Sd}}{\sin\alpha} \quad (6.3\text{–}24)$$

$$R_{Rtw} = \left[\frac{A_{sw} f_{yd}}{s}\right] z(\cot\theta + \cot\alpha) \quad (6.3\text{–}25)$$

To optimize the shear design, in the sense of reducing the amount of stirrups required, having calculated σ_1, the available σ_2 can be evaluated by means of eqs (6.3–20) and (6.3–23). Then the corresponding θ can be obtained from eq. (6.3–22) and can be used in eq. (6.3–25) to determine the stirrups required for $F_{Rtw} \geqslant F_{Stw}$.

6.3.3.3.4. Simple approach
This approach is of general applicability. Model 1 is that of Fig. 6.3.10(b) which contains no arch. Thus in effect the design values of all the forces due to prestress and all other actions are applied to the truss model of Fig. 6.3.10(c).

The forces due to prestress are

- the end moments and shears due to hyperstatic prestress
- the end forces (M, N, V) due to isostatic prestress
- the curvature effects of the tendons throughout the zone under consideration.

6.3.3.3.5. Special conditions

Tension chord near beam ends/end zones of pretensioned members

The distribution of force along the tension chord can be calculated from the models adopted.

In the case of a pretensioned member, in which the transmission length extends into the span, the safety of the end zone with respect to the forces corresponding to model 1 of clauses 6.3.3.3.2 and 6.3.3.3.3 can be assured only if the concrete there remains uncracked under the design action effects, i.e. if the principal tensions at both the centroid and the extreme fibre (calculated for an uncracked section) remain below $f_{ctk,\min}/1.5$. If this condition is complied with, the end zone's deviation from simple arching can be accepted.

If this condition is not complied with the approach of clause 6.3.3.3.4 should be adopted.

Intermediate anchorages

In the case of several tendons having different profiles/locations of anchorages, separate funicular systems may be considered, generating a number of models type 1, the stress fields of which are to be superimposed upon one another.

If intermediate anchorages are made at or near the compressed face all the effects from the tendons in question should be applied to model 2.

Inversion of tendon curvature

In the case of inversion of tendon curvature, the direct effects of downward curvature are to produce additional transverse loading and are thus unfavourable. Model 1 can none-the-less be such that it still resists a constant fraction of the loading and the approach of clause 6.3.3.3.3 remains applicable.

Within the transmission zone, the bond stresses of the tendons may increase beyond their values at transfer, by virtue of the effect of transverse pressure from a reaction, but in many cases supplementary passive reinforcement is required to provide for the change of longitudinal force within the transmission zone.

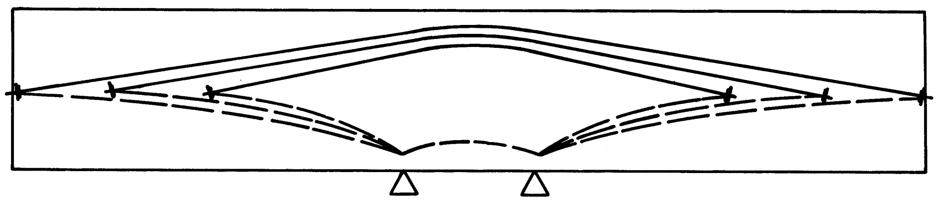


Fig. 6.3.12. Intermediate anchorages away from the compressed face

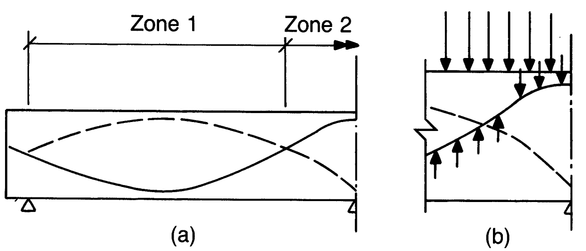


Fig. 6.3.13. Inversion of tendon curvature (possibility of increasing arch slope shown in (b))

Suspended loads

In the descriptions (clauses 6.3.3.3.1 to 6.3.3.3.3) of model 1 it is assumed that the loading is applied in such a manner as to create transverse compression within the member. If this is not the case additional transverse reinforcement is required to 'suspend' the loads.

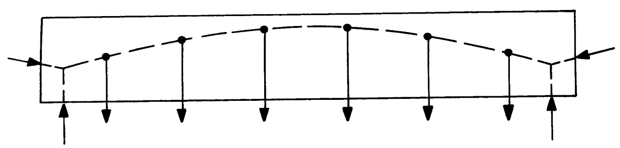


Fig. 6.3.14. Additional stirrups for suspended loads

6.3.3.4. Columns subjected to axial load, bending and shear

For beam-column joints, see clause 6.8.2.2.

6.3.3.4.1. General

The range of models which might represent the detailed behaviour of columns is very large on account of the range of possible loading conditions and the range of proportions between the longitudinal forces resisted by concrete in compression, steel in compression and steel in tension.

Appropriately precise and consistent models should be used but simplifications are possible and two simple approaches (for different conditions) are given in clauses 6.3.3.4.2 and 6.3.3.4.3.

In all cases the following conditions should be respected.

It should be noted that the design stresses for the reinforcement reach design yield values only if the strains according to subsection 6.3.2 attain their yield values.

- The sections at the ends of the column length considered should be verified vs. axial action effects using the methods of subsection 6.3.2.
- In the case of slender columns (see section 6.6), where sections affected by second order moments are critical these sections should also be verified using the methods of subsection 6.3.2.
- The column should be provided with at least minimum links according to subsection 9.2.3.
- Minimum longitudinal reinforcement should be provided in accordance with subsection 9.2.3.

The two methods given below are intended for use in different situations:

- clause 6.3.3.4.2 where the compression loading is dominant and longitudinal tension reinforcement is not required
- clause 6.3.3.4.3 where the compression is less dominant.

6.3.3.4.2. Design of columns in which compression is dominant

Where the compression loading is so dominant that no longitudinal tension reinforcement is required in any part of the length in question, then it should be checked whether or not inclined cracking is to be expected under the design action effects.

For this purpose the longitudinal normal stress and transverse shear stress in the concrete may be calculated approximately as

$$\sigma_n = N_{Sd}/A_c \tag{6.3–26}$$

N_{Sd} here is taken positive for compression.

where A_c is the cross-sectional area.

$$\tau = \frac{3V_{Sd}}{2b_w h} \text{ for rectangular cross-sections} \tag{6.3–27}$$

where

b_w is the web breadth
h is the overall dimension in the direction of V_{Sd}.

Approximate values of the principal stresses may be calculated as

tension: $\sigma_{c1} = \sqrt{(\tau^2 + \sigma_n^2/4)} - \sigma_n/2$

compression: $\sigma_{c2} = \sqrt{(\tau^2 + \sigma_n^2/4)} + \sigma_n/2 \leqslant f_{cd1}$.

Inclined cracking may be assumed to be avoided if

If σ_{c1} is larger than the limiting value of (a) or (b), then clause 6.3.3.4.3 should apply.

(*a*) for $\sigma_{c2} < f_{cd}/3$:

$$\sigma_{c1} \leqslant f_{ctk,\min}/1.5$$

(*b*) for $\sigma_{c2} \geqslant f_{cd}/3$:

$$\sigma_{c1} \leqslant f_{ctk,\min}\left(1 - \frac{\sigma_{c2}}{f_{cd}}\right)$$

In this case only the nominal transverse reinforcement defined in subsection 9.2.3 is required.

6.3.3.4.3. Design of columns in which compression is less dominant
If longitudinal tension reinforcement is required in any part of the length of the column, the differentiated modelling approach of clause 6.3.3.3.2 is followed, appropriately modified to meet the boundary conditions of the column.

In the case of a building column as in Fig. 6.3.15, the following models may be superimposed

- model 1, in which the only forces acting on the column are end compressions N_c, numerically equal to the lesser of the two compressive forces in the concrete obtained from the designs of the end sections for

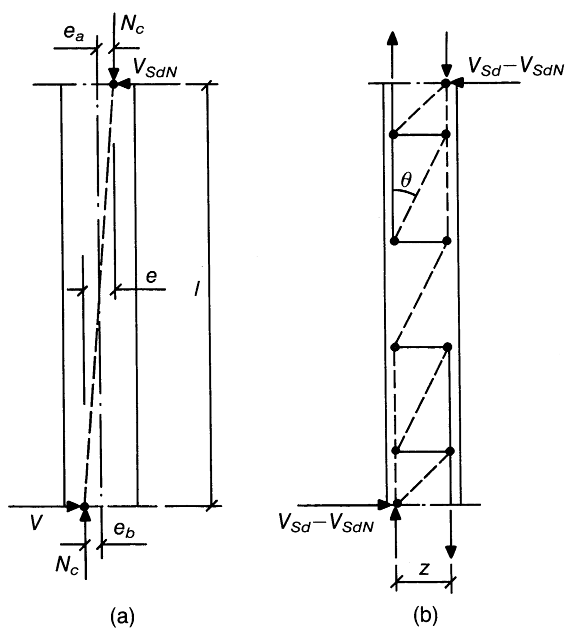


Fig. 6.3.15. Models for columns with less dominant compression: (a) model 1; (b) model 2

axial action effects, and shears V_{SdN} needed to deviate the direction of N_c in order to form a diagonal strut across the whole column

- model 2, in which the remaining axial action effects act on the ends, together with the shear forces $V_{Sd} - V_{SdN}$. A truss model represents the internal actions.

The stress fields of the two models are superimposed for the final verifications of the web (and of the curtailment of longitudinal steel if relevant).
The compression of the web concrete is verified from the following

Acting stress

$$\sigma_{Scw} = \sigma_1 + \sigma_2$$

$$\sigma_1 = \frac{N_c}{b_w z}\sqrt{\left(\frac{l^2 + e^2}{l^2}\right)} \qquad (6.3\text{–}28)$$

where e is the sum of the eccentricities of N_c at the two ends

$$\sigma_2 = \frac{V_{Sd} - V_{SdN}}{b_w z \sin\theta\cos\theta} \qquad (6.3\text{–}29)$$

Resistant stress

$$\sigma_{Rd} = f_{cd2}$$

The tension of the stirrups (assumed to be perpendicular to the axis of the column):

$$F_{Stw} = V_{Sd} - V_{SdN}$$

$$F_{Rtw} = \frac{A_{sw} f_{yd}}{s} z \cot\theta$$

6.3.4. Longitudinal shear in flanged sections

The reinforcement needed to transmit the longitudinal shear in flanged sections may be determined by means of truss models such as those in Fig. 6.3.16.

Along the axis of the beam, the distribution of the longitudinal shear between the web and the flange should be determined from the truss model used for the web.

The longitudinal shear V per unit length of the axis of the beam is determined by the change in the normal (longitudinal) forces in the actual part of the flange:

$$V = \Delta F / \Delta x \tag{6.3–30}$$

where

Δx is the length under consideration

ΔF is the change in the normal force in the actual part of the flange in the length Δx.

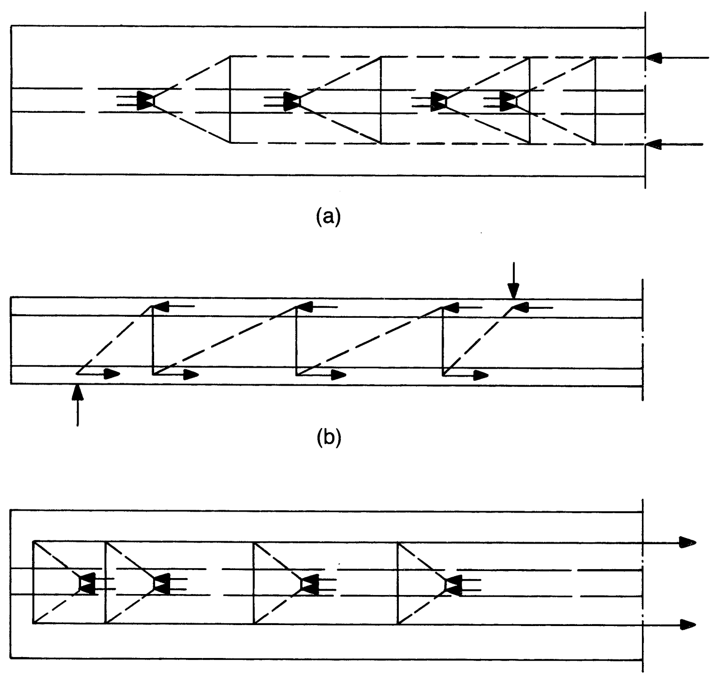


Fig. 6.3.16. Truss models for an I-beam: (a) compression flange; (b) web; (c) tension flange

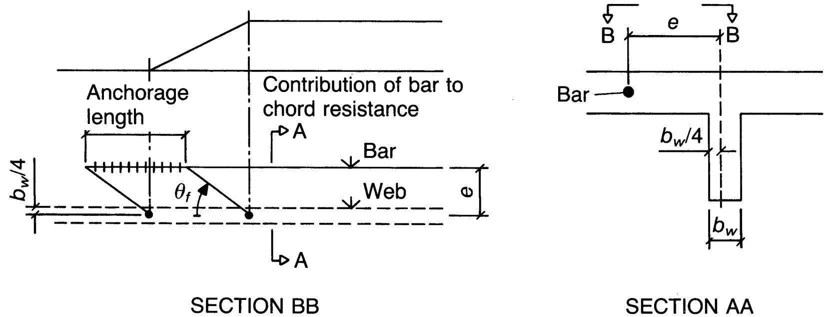
The transverse reinforcement per unit length A_{sf} may be determined by the equation

$$A_{sf} f_{yd} \geqslant V \tan \theta_f \tag{6.3–31}$$

As a simplification the following values of $\tan \theta_f$ may be used

— $\tan \theta_f = 0.5$ for compression flanges
— $\tan \theta_f = 0.8$ for tension flanges.

At any given section of a beam, the curtailment of a bar placed in the tension flange should be appropriately determined. To this purpose, the anchorage length of the bar should be sufficiently ahead of the section where its resistance is required. If e is the distance between the axis of the bar and a line situated at the quarter point of the web breadth, the anchorage length of the bar should end at least $e \cot \theta_f$ ahead of the given section (Fig. 6.3.17).



6.3.5. Torsion

6.3.5.1. Scope and basic assumptions

The following sections apply to linear members subjected to torsion combined with axial action effects and shear.

They apply to solid cross-sections and to hollow or open sections in which the effects of longitudinal shear and transverse bending are negligible. These effects may be negligible due to the thickness of the walls of the section or to the loading conditions.

For methods of verification for members such as hollow box girders where the effects of longitudinal shear are not negligible reference should be made to section 6.5.

In the case of T-beams, if the torsion is assumed to be resisted only by the web, the web may be designed according to this section and the flange according to subsection 6.3.4.

(*a*) A distinction may be drawn between

(i) equilibrium torsion, in which the torque is necessary for equilibrium

(ii) compatibility torsion, in which the torque is due solely to the restraint of rotation induced by adjoining members.

In the case of compatibility torsion, the torsional moments may be neglected in the calculation for the ULS provided the member is reinforced with closed stirrups perpendicular to the axis, and

(i) the stirrups have their legs close to the boundaries of the section

(ii) the stirrups provide a value of $\rho_{sw} = A_{sw} f_{yk} / (b_w s f_{ctm}) \geqslant 0.2$ where A_{sw} is the area of two legs of a stirrup

(iii) the spacing s of the stirrup legs does not exceed the lesser of $0.75b$ and $0.75d$ in the longitudinal direction or $0.75d$ in the transverse direction.

If compatibility torsion is neglected, the neglect should be consistent in the analysis and member design. Thus torsional stiffness should be taken equal to zero in the analysis.

If the analysis is not based on the neglect of torsion a realistic stiffness should be adopted (see section 3.8) and the ULS verification should take account of the torque obtained from the analysis.

(*b*) A distinction may also be drawn between

(i) circulatory torsion, in which equilibrium is maintained by a closed flow of tangential shear

(ii) warping torsion, due to a restraint of longitudinal deformations.

Resistance to torsion is provided by shear forces, which may or may not require longitudinal bending actions. Torsional resistance without longitudinal bending corresponds, in terms of elastic theory, to St. Venant torsion. It is here described by the more general term 'Circulatory torsion' (Fig. 6.3.18).

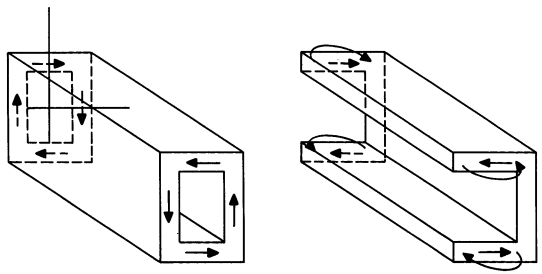


Fig. 6.3.18. Circulatory- and warping torsion

(*c*) Away from local disturbances at supports or sections subjected to concentrated loads etc., the action effects due to torsion at any section may be treated as longitudinal moments and shear forces acting on actual or equivalent walls representing the member. Each wall may then be designed for the summed effects of torsion and other load effects.

(*d*) In the verification methods given in clauses 6.3.5.2 and 6.3.5.3 it is assumed that the member is treated as containing inclined cracks and that its walls are designed on the basis of truss models.

There is no absolute division between circulatory and warping actions. The main part of a hollow box girder resists torsion by a circulatory action, but warping effects are produced near diaphragms. An open C-shape may resist torsion primarily by warping but its component rectangles do develop some circulatory torsion. For many members it is sufficient to design for the primary mode of resistance assuming it to carry the total torsion. For major members it is, however, possible to assess a division between circulatory and warping effects.

6.3.5.2. Circulatory torsion

(a) Hollow polygonal convex cross-sections

The effective thickness t_{ef} taken into account in the calculations shall not

- exceed the actual wall thickness
- be less than twice the distance between the external face of the wall and the line joining the axes of the longitudinal reinforcement.

Subject to the above limitation, the minimum effective thickness required may be determined from consideration of the combined effects of M, N, V and T.

As a first approximation, the effective thickness may be assumed as

- the actual wall thickness if this thickness is less than (A/u), or

$$t_{ef} = A/u \text{ in the opposite case}$$

where

u is the external perimeter of the cross-section
A is the area limited by this perimeter.

In the flexural tensile zone advantage may be taken of the increased lever arm (for flexure) if the longitudinal reinforcement is placed outside the centre of the wall thickness adopted for tension.

In a solid rectangular cross-section the effects of M, V and N may be considered to act over the full breadth of the section, but may be redistributed to optimize the design. For example either of the two solutions shown in Fig. 6.3.19 for the summation of the effects of V and T is acceptable.

In a similar manner the effects of N may be distributed over the section in proportion to the areas of its parts, may be distributed to all of the walls or may be distributed to only some of the walls.

(b) Solid sections

A polygonal convex solid section may be verified as an equivalent hollow section in which the thickness of the 'walls' shall not be less than twice the distance between the external face and a line joining the axes of the longitudinal reinforcement. As a first approximation the effective thickness may be assumed as $t_{ef} = A/u$.

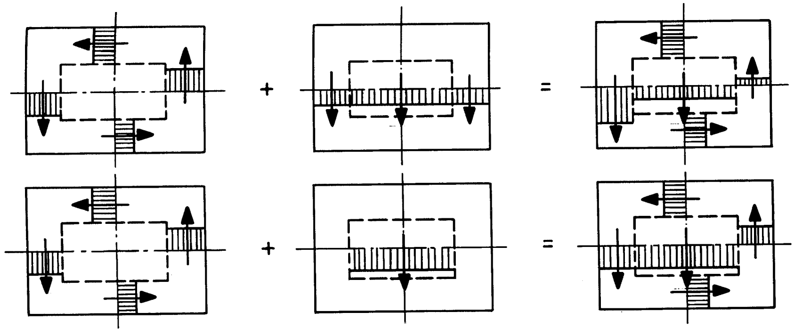


Fig. 6.3.19. Alternative summations of torsion and shear

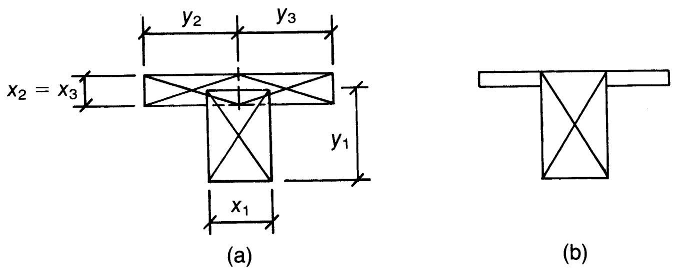


Fig. 6.3.20. Treatment of sections composed of rectangles: (a) division into component rectangles; (b) use of a single main rectangle

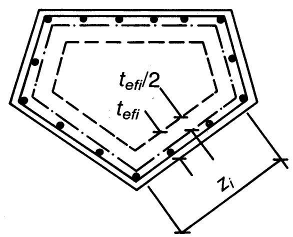


Fig. 6.3.21. Notation for eqs (6.3–32) and (6.3–33)

(c) Sections composed of rectangles

For sections composed of rectangles the applied torque may be assumed to be distributed between the rectangles in proportion to the values of $x_i^3 y_i$ for each of them

$$T_{Sdi} = T_{Sd} \frac{x_i^3 y_i}{\Sigma x_i^3 y_i}$$

where

T_{Sdi} is the torque assumed to be resisted by the ith rectangle
x_i is the smaller dimension of the ith rectangle
y_i is the larger dimension of the ith rectangle.

Each rectangle i is then treated as a solid section subjected to the torque T_{Sdi}.

If one of the rectangles has a value of $x_i^3 y_i$ markedly geater than those for the other rectangles then this rectangle should be assigned the entire torque.

SHEAR FLOW AND SHEAR FORCE

In a hollow polygonal convex cross-section (actual or equivalent) the shear flow due to a torque T_{Sd} may be assumed as constant and acting at the centre of each wall.

With

t_{efi} the effective thickness of the ith wall
z_i the distance, for the ith wall between the intersections of its centre-line with those of adjacent walls (see Fig. 6.3.21)

the shear flow is

$$\tau_{ti} t_{efi} = T_{Sd}/2A_{ef}\delta \qquad (6.3\text{–}32)$$

and the shear force in the ith wall is

$$V_{Sdi,t} = T_{Sd} z_i / 2A_{ef}\delta \qquad (6.3\text{–}33)$$

where

τ_{ti} is the shear stress due to torsion
A_{ef} is the area encosed by the centre-lines of walls
δ is a numerical coefficient.

For a circular section $\delta = 1.0$.

For a rectangular section of side dimensions b_x and b_y, where $b_y > b_x$

$$\delta = 1.0 - 0.25 b_x / b_y \qquad (6.3\text{–}34)$$

The coefficient δ allows for model imperfections/uncertainties probably related to the flow of stress around corners.

ACTION EFFECTS TO BE CONSIDERED

Each wall i should be designed for the shear and normal forces due to M_{Sd}, N_{Sd}, V_{Sd} and T_{Sd} (see Fig. 6.3.22)

$$V_{Sdi} = V_{Sdi,t} + V_{Sdi,v} \qquad (6.3\text{–}35)$$

where

$V_{Sdi,t}$ is the shear force due to torsion, see eq. (6.3–33)
$V_{Sdi,v}$ is the shear force due to transverse shear

$$N_{Sdi} = N_{Sdi,m} + N_{Sdi,n} \qquad (6.3\text{–}36)$$

where

$N_{Sdi,m}$ is the longitudinal force due to flexure
$N_{Sdi,n}$ is the longitudinal force due to axial load.

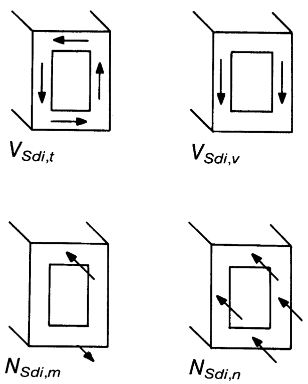


Fig. 6.3.22. Shear and normal forces on walls

The effects of prestress should be taken into account in the calculation of V_{Sdi} and N_{Sdi}. The prestressing forces should be treated as an external loading system with safety factors as described in clause 1.6.2.4 a1.

VERIFICATIONS

The verifications differ depending on whether or not the wall is assumed to

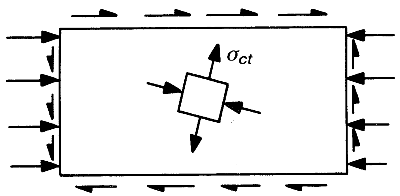


Fig. 6.3.23. Uncracked wall

(*a*) Uncracked wall
A wall may be treated as uncracked if

- it is parallel to the axis about which the member is bent by flexure, and
- the maximum principal stress $\sigma_{ct,\max}$ resulting from σ_{Sdi} and τ_{Sdi} is less than or equal to f_{ctd}, where $f_{ctd} = f_{ctk,\min}/1.5$ and $f_{ctk,\min}$ is given in Table 2.1.2.

Where a wall contains ducts or bars (see clause 6.2.2.4).

It is not prohibited to use different depths of main compression zone for the effects of torsion and flexural/axial load. In this case the relevant areas should be used separately to obtain stresses from forces.

The verification ($\sigma_{ct,\max} \leqslant f_{ctd}$) should be made at the centre of the wall and the shear stresses due to torsion should be estimated from eq. (6.3–32), while the stresses due to the remaining load effects should be calculated on the assumption of linear elastic behaviour.

If the condition is satisfied, in addition the principal compressive stress resulting from σ_{Sdi} and τ_{Sdi} should be less than f_{cd1}.

(*b*) Wall with inclined cracks
The following verifications should be made if either of the conditions in (*a*) is not satisfied.

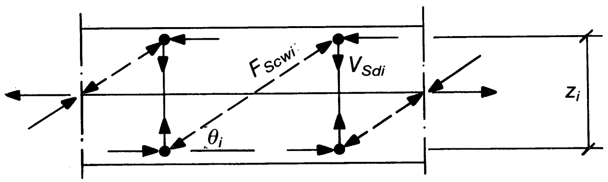


Fig. 6.3.24. Truss model of a wall

The model of Fig. 6.3.24 may be used so long as the following provisions are respected.

- The spacing of stirrups should not exceed $u_s/8$ where u_s is the perimeter of the stirrups.
- The stirrups should provide effective continuity from wall to wall.
- At each junction of walls, there should be a longitudinal bar with a diameter at least equal to $s/16$ where s is the spacing of the stirrups.

The angle θ_i between the wall compression and the longitudinal direction may be chosen freely in the range 18° to 45°.

The forces derived from the model of Fig. 6.3.24 and the verifications required are as follows

1. Longitudinal force

$$F_{Sli} = N_{Sdi} + V_{Sdi} \cot \theta_i \qquad (6.3\text{–}37)$$

with tension taken positive.

For $f_{pyd,net}$ see subsection 6.2.4.

$$F_{Rli} = A_{Si} f_{yd} + A_{pi} f_{pyd,net} \quad (6.3\text{–}38)$$

The reinforcement required for F_{Rli} should generally be distributed over the length of z_i, but for smaller sections it may be concentrated at the ends of this length.

If $F_{Sli} < 0$ no separate verification is required provided that the wall is treated as cracked and its inclined compression is verified as in 2 below.

2. Inclined compression of wall concrete

$$F_{Scwi} = V_{Sdi}/\sin\theta_i \quad (6.3\text{–}39)$$

$$F_{Rcwi} = f_{cd2}\, t_i z_i \cos\theta_i \quad (6.3\text{–}40)$$

3. Tension of transverse reinforcement

$$F_{Stwi} = V_{Sdi} \quad (6.3\text{–}41)$$

$$F_{Rtwi} = A_{Swi} f_{yd} \cot\theta_i \frac{z_i}{s} \quad (6.3\text{–}42)$$

where A_{Swi} is the area of one unit of shear reinforcement in the ith wall, e.g. one leg of a stirrup in a solid section or two units of reinforcement (one at the inner and one at the outer face) in a wall of a box girder.

Warping torsion is found predominantly in linear members with thin open cross-sections formed of at least three walls.

The reduction of warping torsion stiffness due to cracking is similar to that of bending stiffness and smaller than the reduction for circulatory torsion.

6.3.5.3. Warping torsion

For open sections having at least three walls in separate planes, the shear forces and bending moments due to torsion in each of the walls should be determined from the requirements of static equilibrium. Each wall may then be designed for the summed effects of torsion and other load effects.

6.4. SLABS

6.4.1. Bending and torsion

Subsection 6.4.2 is applicable also to prestressed concrete slabs.

In a region of a slab primarily subjected to moments m_x and m_y parallel to the directions of the reinforcement the design for bending should follow subsection 6.3.2 and design for shear should follow subsection 6.4.2.

In more general cases slabs are subjected to moments m_x, m_y and m_{xy} per unit width. The design should then be based upon a model in which the outer layers resist the in-plane effects of the moments and the inner layer

Prestressed slabs may be verified as plates subjected to in-plane compression and transverse loading (see subsection 6.5.4).

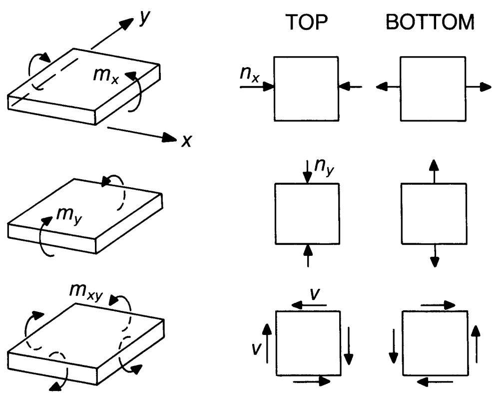


Fig. 6.4.1. Forces in the outer layers of a slab due to moments m_x, m_y, m_{xy}*: if the moments are in reversed directions, the in-plane forces have reversed signs*

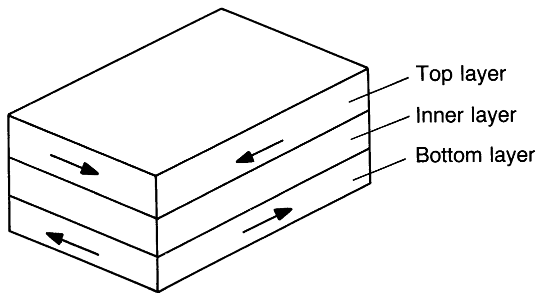


Fig. 6.4.2. Example of layer forces in a slab resisting torsion

At free edges of slabs the set of reinforcement perpendicular to the edge, having the greater area per unit width, should be bent into the plane of the edge and returned into the other layer to provide torsional resistance. Alternatively U-bars may be inserted at the edge and lapped with the top and bottom steel.

As illustrated by Fig. 6.4.1 the forces per unit width acting on the outer layers are

$$n_{Sdx} = m_{Sdx}/z_x \tag{6.4–1}$$

$$n_{Sdy} = m_{Sdy}/z_y \tag{6.4–2}$$

$$v_{Sd} = m_{Sdxy}/z_{xy} \tag{6.4–3}$$

with signs as determined by the signs of the moments.

z_x is the internal lever arm between the tensile and compressive normal forces in the x-direction

z_y is the internal lever arm between the tensile and compressive normal forces in the y-direction

z_{xy} is the internal level arm between the shear forces of the upper and lower layers.

As an approximation z_{xy} may usually be taken as $2h/3$, where h is the overall thickness of the slab.

It should be noted that in the general case the dimension z_x or z_y is the distance between the centres of the resultants of compression in the concrete and tension in the reinforcement. The dimension z_{xy} is always between the centres of concrete forces.

The criteria for ULS are given in subsection 6.5.3.

It results from sections 1.3 and 1.4.5, that this possible deviation should not be taken less than the greater of

- 1.2 times the tolerance
- 2.4 times the expected standard deviation (possibly subjectively assessed).

If the thickness of a slab is smaller than 100 mm, the possible deviation of the depth d of the reinforcement, exceeding 5% of the nominal value of d should be taken into account in the calculations.

6.4.2. Transverse shear distributed over the width of a slab

The methods of verification given in this section are concerned with failures occurring more or less immediately upon the formation of major shear cracks and the resulting breakdown of normal beam action. In short shear spans where loads and reactions act in such a way as to produce transverse compression an enhanced resistance may be possible due to an arch action — see the final part of clause 6.3.3.2.

6.4.2.1. Scope

The following paragraphs apply to solid slabs in regions subjected to distributed loads. They also apply to slab-like members such as hollow-core units, the T-beams of ribbed or coffered slabs and precast units so connected as to form a slab-like structure.

They may further be applied to linear members of minor importance such as lintels.

Where a slab spans in one direction parallel to reinforcement the verification of its shear resistance is made in terms of the applied shear on sections normal to the span and of the shear resistance associated with the reinforcement in the direction of the span. This sort of verification is treated in clauses 6.4.2.2 to 6.4.2.4 dealing respectively with uncracked zones, cracked zones without shear reinforcement and slabs with shear reinforcement.

See the end of this paragraph for special recommendations for the end regions of prestressed elements.

6.4.2.2. Shear in uncracked zones

If the zone in question should not be cracked under the ULS action effects, it should be verified that the maximum principal tensile stress satisfies the inequality

$$\sigma_{ct,\max} < f_{ctd} \qquad (6.4\text{–}4)$$

where

$f_{ctd} = f_{ctk,\min}/1.5$
$f_{ctk,\min}$ is given in Table 2.1.2.

In principle the verification should be carried out at all sections and at all levels. However as a simplification, for rectangular I, or T-sectioned members spanning in one direction and having their centroids within the web, it is sufficient to perform the calculation in terms of a nominal shear stress $\tau = 3V_{Sdc}/(2b_w h)$ and the design longitudinal stress at the centroid.

The applied shear on the concrete (V_{Sdc}) should be calculated taking

where the effects are favourable and the higher ones where they are unfavourable.

The presence of prestressing ducts should be taken into account by subtracting from the web breadth the sum of the diameters of the ducts in one layer.

If the centroid of the cross-section does not lie within the web the principal tensile stress verification should be made at the intersection of the web and flange within which the centroid lies.

In pretensioned members account should be taken of the reduction of the effective prestress within the transmission lengths.

Where the centroid of the section is at mid-depth the reduction may be allowed for by making the calculations for a section distance $h/2$ from the inner edge of the support and taking the prestress at the centroid as

$$\sigma_c = \frac{P_{gd}}{A}\left(1 - \frac{s + 0.5h}{l_{bpt}}\right) \qquad (6.4\text{–}5)$$

where

P_{gd} is the design prestressing force under permanent loads
A is the area of the section
s is the distance from the end of the member to the inner edge of the supports
l_{bpt} is the transmission length.

l_{bpt} is determined according to clause 6.9.11.4 with $\alpha_9 = 1.0$.

6.4.2.3. Shear in cracked zones without shear reinforcement

The net shear forces acting on the concrete should be taken as

(*a*) for ordinary reinforced concrete members, the total shear $V_{Sdc} = V_{Sd}$
(*b*) for prestressed members, the shear corresponding to the part of the external loading system not balanced by the prestress system

$$V_{Sdc} = V_{Sd} - V_{\lambda} \qquad (6.4\text{–}6)$$

where V_{λ} is the shear corresponding to λq_{sd} determined as in clause 6.3.3.3.3.

It is to be verified that

$$V_{Sdc} \leqslant V_{Rd1} \qquad (6.4\text{–}7)$$

The transverse shear resistance V_{Rd1} may be described by means of an appropriate model.

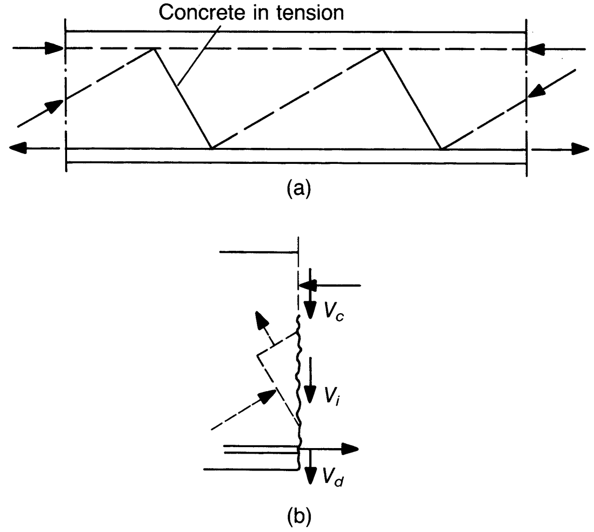


Fig. 6.4.3. Shear resistance of members without shear reinforcement: total shear = $V_c + V_i + V_d$, *where* V_c *is shear in compression zone,* V_i *is shear carried across cracks by concrete-to-concrete contact, and* V_d *is shear carried across cracks by dowel action of reinforcement*

Wherever a more precise analysis is not made, the following empirical expression may be used for members with parallel chords:

$$V_{Rd1} \quad = \quad 0.12\xi(100\rho f_{ck})^{1/3} b_{red} d \qquad (6.4\text{–}8)$$

where

$\xi = 1 + \sqrt{(200/d)}$ with d in mm

ρ is the ratio of bonded flexural tensile reinforcement $A_s/b_w d$ or $(A_s + A_p)b_w d$ extending for a distance at least equal to d beyond the section considered, except at end supports where the extension may be considered adequate if the length of bar beyond the centre-line of support is equal to at least 12 times the diameter

b_{red} is the reduced web breadth equal to the full breadth minus the sum of the widths of tendon ducts situated within the web (note no deduction is necessary for ducts at the boundary of the web, i.e. at the level of the main tension reinforcement).

Fig. 6.4.3 illustrates indicative models of the actions resisting shear.

For high concrete strengths the relationship between the shear resistance of a member and the strength of the concrete depends upon the characteristics of the aggregate. If the aggregate fractures at flexural cracks, leaving smooth crack surfaces, the shear resistance may be below that given by eq. (6.4–8) unless f_{ck} is restricted. Relevant experimental evidence may be provided by tests of beams of concrete with the intended value of f_{ck} and the aggregate of the type to be used.

Unless relevant experimental evidence is available for the concrete in question, f_{ck} should be limited to 50 MPa for the purpose of calculation according to eq. (6.4–8).

Except at simple supports, flexural tensile reinforcement should extend at least $0.6d$ beyond the section at which it is no longer required according to flexural calculations.

In the case of a member subjected to axial tensile loading the reinforcement required to resist this load should be discounted in the calculations of ρ.

Bonded prestressing tendons can be included in the calculation of ρ but unbonded tendons should be excluded.

6.4.2.4. Shear in slabs with shear reinforcement

The verification of the resistance of a slab with shear reinforcement should

be made in accordance with subsection 6.3.3, but the general requirements of clause 6.3.3.1 may be relaxed as follows

(*a*) The minimum shear reinforcement, giving a value of $\omega_{sw} = A_{sw}f_{yk}/(b_w s f_{ctm})$ of at least 0.2, need be provided only where the applied shear exceeds the shear resistance of the slab without shear reinforcement, and for a distance equal to d in the direction of decreasing shear. The minimum shear reinforcement need not be in the form of stirrups.

(*b*) The inclination of stirrups and bent-up-bars to the axis should be at least 45° and 30° respectively as for linear members.

(*c*) The spacing of members of shear reinforcement in the longitudinal direction should not exceed $0.75d(1 + \cot\alpha)$, where α is the inclination of the shear reinforcement.

(*d*) Longitudinal reinforcement is not required to be contained within a stirrup cage but the shear reinforcement should be anchored at the level of the flexural tensile reinforcement and at the level of the centre of the flexural compression force in the ULS.

It should be noted here, that even in the ULS the depth of the compression zone is the elastic one except at the section of maximum moment.

6.4.2.5. General cases of two-way spanning slabs

Where a slab spans in two directions, the results of the analysis include shears v_x and v_y per unit width. The principal shear is then

$$v_1 = \sqrt{(v_x^2 + v_y^2)} \qquad (6.4\text{–}9)$$

and acts on a surface at an angle ϕ to the y-axis, where

$$\phi = \arctan(v_y/v_x) \qquad (6.4\text{–}10)$$

The shear on a perpendicular surface is zero.

The verification should be made with respect to the principal shear and, for an ordinary reinforced concrete slab, it is required that

$$v_1 b \leqslant V_{Rd1} \qquad (6.4\text{–}11)$$

where

V_{Rd1} is given by eq. (6.4–8) with d taken as the mean effective depth

$$\rho = \rho_x \cos^4\phi + \rho_y \sin^4\phi \qquad (6.4\text{–}12)$$

ρ_x and ρ_y are the ratios of reinforcement near the face in tension in the direction perpendicular to the surface on which v_1 acts.

Clause 6.4.2.5 is intended to be applied in cases where neither v_x nor v_y is clearly the major shear.

Eq. (6.4–12) is related to the stiffness of the reinforcement in relation to tension in the direction perpendicular to the surface on which v_1 acts.

6.4.3. Concentrated loads on slabs/slab-column connections

6.4.3.1. General

The resistance to the transverse effects of concentrated forces (loads or reactions) acting on slabs without shear reinforcement may be verified in terms of nominal shear stresses at control perimeters.

Provided that the concentrated force is not opposed by a high distributed pressure, e.g. soil pressure on a base, or by the effects of a load or reaction within a distance equal to 2.0d from the periphery of area of application of the force, the control perimeter (u_1) may be taken to be at a distance 2.0d from the above periphery and should be constructed so as to minimize its length.

The effective depth of the slab is assumed constant and may normally be taken as

$$d_{ef} = (d_x + d_y)/2 \qquad (6.4\text{–}13)$$

where d_x and d_y are the effective depths of the reinforcement in two orthogonal directions.

If the slab contains a drop panel around a column one verification should be made for a perimeter 2.0d from the column with d taken as the effective depth within the area of the drop.

A second verification should be made for the area outside the drop panel and for this the smaller slab thickness should be used. If the drop panel is large in area it may be more appropriate to make this second verification in accordance with clause 6.4.2.3.

The verification in terms of the perimeter u_1 is subject to an upper limit in terms of shear stresses at the periphery of the area to which the force is applied.

In the following subsection a few typical cases are presented and simplified approaches to the analysis of stress distributions are given. For more complex cases, e.g. elongated supports, an appropriate structural analysis should be performed.

The normal shear stress on a section defined by a control perimeter does not have any physical meaning, but this empirical approach gives a close approximation to the resistance obtained by mechanical models of punching behaviour.

In reality, for the axisymmetric case, the failure surface is trumpet-shaped/conical. It runs from the edge of the loaded area through the slab to the opposite face at a mean inclination of about 25° to 30° (see Fig. 6.4.4). The inclined crack is already formed at 1/2 to 2/3 of the failure load without leaving the slab in an unstable condition. The resistance thus depends primarily on the conditions of stress and strain in the concrete in the region close to the column.

The analytical methods, the results of which show good approximation to the empirical treatment, include the Kinnunen/Nylander model and the upper bound solution according to plastic theory.

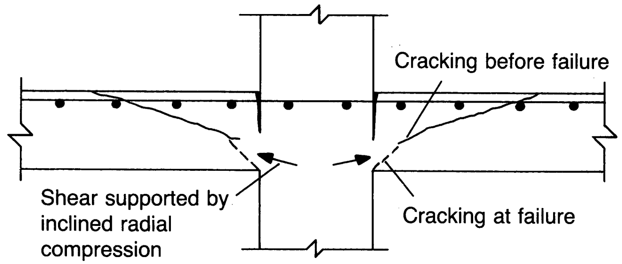


Fig. 6.4.4. Section through a punching failure

The upper limit, which is rarely the governing criterion, is given in clause 6.4.3.4.

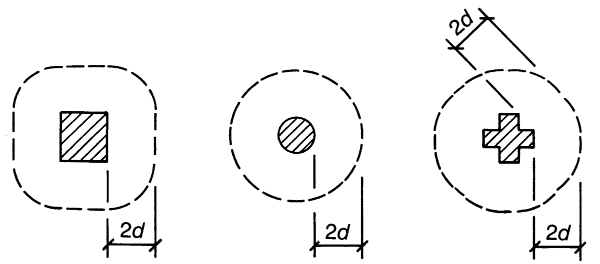


Fig. 6.4.5. Control perimeters at interior columns

6.4.3.2. Stresses due to applied loads

(a) Symmetric loading

If the dispersion of the concentrated force is approximately polar-symmetric the applied shear stress at the control perimeter may be taken as

$$\tau_{Sd} = F_{Sd}/u_1 d \qquad (6.4\text{–}14)$$

where

F_{Sd} is the concentrated force
u_1 is the length of the control perimeter.

(b) Slab-internal column connections transferring moments

If the dispersion of the force is non-symmetrical due to the transfer of an unbalanced moment M_{Sd} from the slab to a column the maximum shear at the control perimeter may be calculated as

$$\tau_{Sd} = \frac{F_{Sd}}{u_1 d} + \frac{KM_{Sd}}{W_1 d} \qquad (6.4\text{–}15)$$

where

W_1 is a parameter of the control perimeter u_1 ($W_1 = \int_0^{u_1} |e| dl$)

dl is an elementary length of the perimeter
e is the distance of dl from the axis about which the moment M_{Sd} acts

K is a coefficient dependent on the ratio between the column dimensions c_1 parallel to the eccentricity M_{Sd}/F_{Sd} and c_2 perpendicular to the eccentricity; its value is a function of the proportions of the unbalanced moment transmitted by uneven shear on the one hand and by bending and torsion on the other.

The property W_1 corresponds to a distribution of shear as illustrated in Fig. 6.4.6.

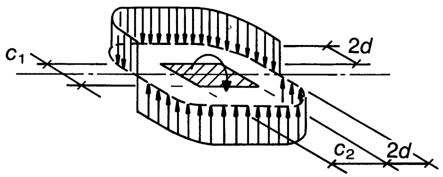


Fig. 6.4.6. Shear distribution due to an unbalanced moment

For a rectangular column

$$W_1 = \frac{c_1^2}{2} + c_1 c_2 + 4c_2 d + 16d^2 + 2\pi d c_1 \qquad (6.4\text{–}16)$$

where

c_1 is the column dimension parallel to the eccentricity of the load
c_2 is the column dimension perpendicular to the eccentricity of the load.

Values of K may be obtained from

c_1/c_2	0.5	1.0	2.0	3.0
K	0.45	0.60	0.70	0.80

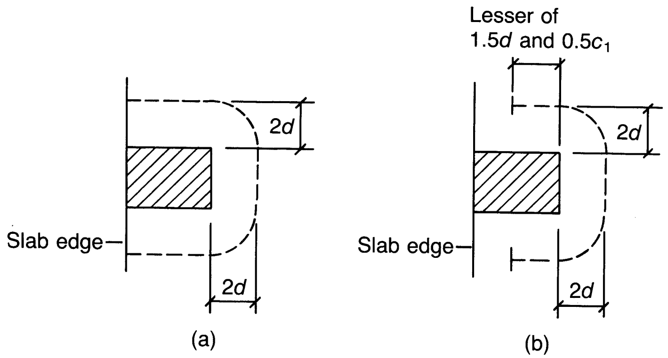


Fig. 6.4.7. Control perimeters at edge columns: (*a*) *perimeter* u_1; (*b*) *perimeter* u_1^*

In such cases the maximum shear is

$$\tau_{Sd} = \frac{P_{Sd}}{u_1^* d} + \frac{KM_{Sd}}{W_1 d} \qquad (6.4\text{–}17)$$

where K may be determined from the table above but with the ratio c_1/c_2 replaced by $c_1/2c_2$. W_1 is calculated for the full perimeter u_1.

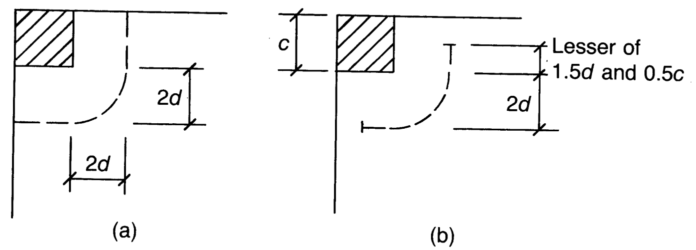


Fig. 6.4.8. Control perimeters at corner columns: (*a*) *perimeter* u_1; (*b*) *perimeter* u_1^*

Unless relevant experimental evidence is available for the concrete in queston, f_{ck} should be limited to 50 MPa for the purpose of calculation according to eq. (6.4–18) (see commentary on clause 6.4.2.3).

(*c*) *Slab-edge column connections*

In principle the distribution of shear around the perimeter in Fig. 6.4.7(a) should be determined to calculate τ_{Sd}.

However, provided the eccentricity of loading in the direction perpendicular to the slab edge is in the direction of the interior of the slab and there is no eccentricity parallel to the edge, τ_{Sd} may be calculated on the assumption of uniform shear on the perimeter u_1^* shown on Fig. 6.4.7(b).

Where a moment M_{Sd} acting in the direction parallel to the slab-edge is transferred to the column it should be taken to produce an additional shear stress equal to $KM_{Sd}/W_1 d$ where W_1 is calculated for the perimeter u_1 of Fig. 6.4.7(a).

(*d*) *Slab-corner column connections*

In principle the distribution of shear around the perimeter in Fig. 6.4.8(a) should be determined to calculate τ_{Sd}.

However, provided the eccentricity of loading is toward the interior of the slab, τ_{Sd} may be calculated on the assumption of uniform shear on the perimeter u_1^* shown on Fig. 6.4.8(b).

6.4.3.3. Resistances of reinforced slabs

The shear resistance of a reinforced concrete slab, expressed as a shear stress on a control perimeter may be taken as

where

$$\xi = 1 + \sqrt{(200/d)} \quad \text{with } d \text{ in mm.}$$

The ratio ρ of flexural reinforcement may be calculated as $\sqrt{(\rho_x \rho_y)}$ where ρ_x and ρ_y are the ratios in orthogonal directions. In each direction the ratio should be calculated for a width equal to the side dimension of the column (or loaded area) plus $3d$ to either side of it (or to the slab edge if this is closer).

6.4.3.4. Maximum resistance

The maximum loading for which any connection (including connections with shear reinforcement and connections involving prestressed slabs) may be designed is defined by

$$F_{Sd,ef}/u_0 d \leqslant 0.5 f_{cd2} \tag{6.4–19}$$

where

$F_{Sd,ef}$ is the punching load enhanced to allow for the effects of an eventual moment transferred to a column

for an interior load or column, u_0 is the length of the periphery of the load or column

for an edge column $u_0 = c_z + 3d \leqslant c_x + 2c_y$

for a corner column $u_0 = 3d \leqslant c_x + c_y$

where for an edge column

c_x is the column dimension parallel to the slab edge

c_y is the column dimension perpendicular to the slab edge.

At an interior column

$$F_{Sd,ef} = F_{Sd}\left[1 + K \frac{M_{Sd}}{F_{Sd}} \frac{u_1}{W_1}\right] \tag{6.4–20}$$

For values of K see clause 6.4.3.2.

At an edge column

$$F_{Sd,ef} = F_{Sd}\left[1 + K \frac{M_{Sd}}{F_{Sd}} \frac{u_1^*}{W_1}\right] \tag{6.4–21}$$

where M_{Sd} is the moment parallel to the slab edge.

At a corner column

$$F_{Sd,ef} = F_{Sd} \tag{6.4–22}$$

6.4.3.5. Verification of column bases

The punching resistances of column bases should be verified at control perimeters at distances up to $2.0d$ from the periphery of the column. The situation at the perimeter giving the lowest column load should be taken to be decisive.

For concentric loading the net applied force is

$$F_{Sd,red} = F_{Sd} - \Delta F_{Sd} \tag{6.4–23}$$

where

F_{Sd} is the column load
ΔF_{Sd} is the net upward force within the control perimeter considered i.e. upward pressure from soil minus self-weight of base

$$\tau_{Sd} = F_{Sd,red}/ud \tag{6.4–24}$$

$$\tau_{Rd} = 0.12\xi(100\rho f_{ck})^{1/3} \times 2d/a \leqslant 0.5 f_{cd2} \tag{6.4–25}$$

where a is the distance from the periphery of the column to the control perimeter in question.

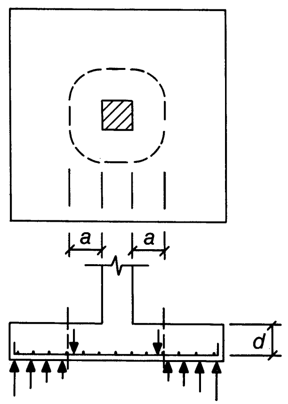


Fig. 6.4.9. Column base

ΔF_{Sd} should be determined taking account of the design (unfavourable) distribution of soil pressure on the base. u is the control perimeter in question ($u \leqslant u_1$).

For eccentric loading

$$\tau_{Sd} = F_{Sd,red}\left[1 + K\frac{M_{Sd}}{F_{Sd}}\frac{u_1}{W_1}\right] \tag{6.4–26}$$

For K see clause 6.4.3.2.

6.4.3.6. Slabs with shear reinforcement

The punching resistances of slabs with shear reinforcement are to be verified in three zones:

- the zone immediately adjacent to the column or loaded area
- the zone in which the shear reinforcement is placed
- the zone outside the shear reinforcement.

At interior slab-column connections subjected to symmetric loading, at edge column connections where there is no eccentricity parallel to the slab edge and the eccentricity perpendicular to it is toward the interior of the slab, and at corner columns where the eccentricity of the reaction is toward the interior of the slab, the verification may be made as follows, subject to the detailing requirements below.

For an interior column u_0 = length of column periphery
for an edge column $u_0 = c_x + 3d \leqslant c_x + 2c_y$
for a corner column $u_0 = 3d \leqslant c_x + c_y$

where for an edge column c_x is the column dimension parallel to the slab edge.

For edge and corner columns, unless an analysis of the shear distribution is made, the perimeters u_1^* should be used for u_1 (see Figs 6.4.7 and 6.4.8).

The ratio ρ of flexural reinforcement may be calculated as $\sqrt{(\rho_x \rho_y)}$ where ρ_x and ρ_y are the ratios in orthogonal directions. In each direction of the column (or loaded area) plus $3d$ to either side of it (or to the slab edge if this is closer).

Anchorage of shear reinforcement in slabs thinner than 250 mm requires special attention to detailing.

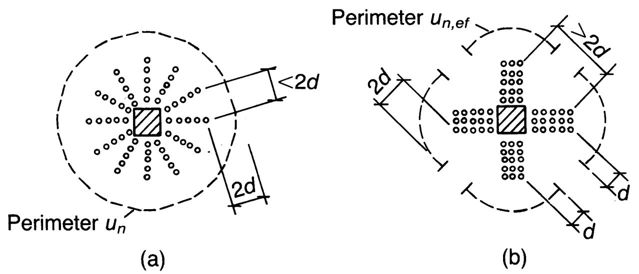


Fig. 6.4.10. Control perimeters (u_n) *at interior columns*

(*a*) *Adjacent to the column*

$$F_{Sd} \leqslant u_0 d(0.5 f_{cd2}) \qquad (6.4\text{–}27)$$

(*b*) *In the zone with shear reinforcement*

$$F_{Sd} \leqslant 0.09\xi(100\rho f_{ck})^{1/3} u_1 d + 1.5 \frac{d}{s_r} A_{sw} f_{ywd} \sin\alpha \qquad (6.4\text{–}28)$$

where

A_{sw} is the area of shear reinforcement in a layer around the column
s_r is the radial spacing of the layers of shear reinforcement
α is the angle between the shear reinforcement and the plane of the slab

$$1.5 \frac{d}{s_r} A_{sw} f_{ywd} \sin\alpha \geqslant 0.03(100\rho f_{ck})^{1/3} u_1 d$$

The design strength of the shear reinforcement (f_{ywd}) should not be taken greater than 300 MPa.

(*c*) *Outside the shear reinforcement*

$$F_{Sd} \leqslant 0.12\xi(100\rho f_{ck})^{1/3} u_{n,ef} d \qquad (6.4\text{–}29)$$

where

$u_{n,ef}$ is the effective length of a perimeter constructed at a distance $2.0d$ outside the outermost shear reinforcement,
ρ is calculated for the reinforcement crossing $u_{n,ef}$.

If the circumferential spacing of the outermost shear reinforcement exceeds $2d$, $u_{n,ef}$ is the sum of the lengths of perimeters corresponding to parts of the periphery of the shear reinforcement within distances d of the elements of the shear reinforcement.

The detailing requirements referred to above are as follows.

(*a*) The distance from the innermost shear reinforcement to the periphery of the column should not exceed βd, where

$$\beta = \frac{\text{capacity of slab without shear reinforcement}}{\text{required capacity}} \leqslant 0.5$$

(*b*) The shear reinforcement should be anchored at or beyond the planes of the tensile reinforcement and centre of flexural compression of the slab.

(*c*) The radial spacing of the shear reinforcement should not exceed $0.75d$.

(*d*) At edge and corner columns, the shear reinforcement required by calculation should be placed within the segments indicated in Fig. 6.4.11. Similar reinforcement at the same spacings should be provided in the areas between these segments and the slab edge or edges, but should not be taken into account in calculations.

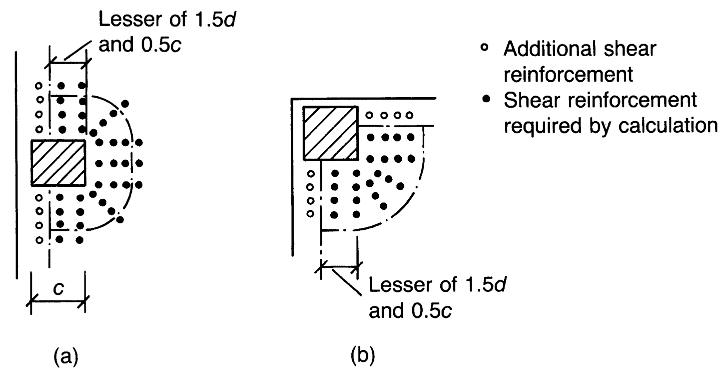


Fig. 6.4.11. Shear reinforcement at edge and corner columns

At interior columns to which moments are transferred and at edge columns where there is eccentricity of loading parallel to the slab edge, the force F_{Sd} should be magnified to $F_{Sd,ef}$ to allow for the influence of the transferred moment.

(*a*) Adjacent to the column and in the zone with shear reinforcement

$$F_{Sd,ef} = F_{Sd}\left[1 + K\frac{M_{Sd}}{F_{Sd}}\frac{u_1}{W_1}\right] \qquad (6.4\text{–}30)$$

(*b*) Outside the shear reinforcement

$$F_{Sd,ef} = F_{Sd}\left[1 + K\frac{M_{Sd}}{F_{Sd}}\frac{u_{n,ef}}{W_{n,ef}}\right] \qquad (6.4\text{–}31)$$

$W_{n,ef}$ is the parameter of the perimeter $u_{n,ef}$ analogous to W_1 for u_1 (see Fig. 6.4.6).
For values of K see clause 6.4.3.2.

The verification may then be made as for a connection without eccentricity of loading and the shear reinforcement should be placed uniformly around the column.

6.4.3.7. Prestressed slabs

For prestressed slabs the sum of the vertical components of the forces in prestressing tendons passing through a column or within a distance $h/2$ of it may be deducted from the load F_{Sd}.

Other influences of prestress may be taken into account by methods given in relevant specialist literature, e.g. the FIP Recommendations for the design of flat slabs in post-tensioned concrete.

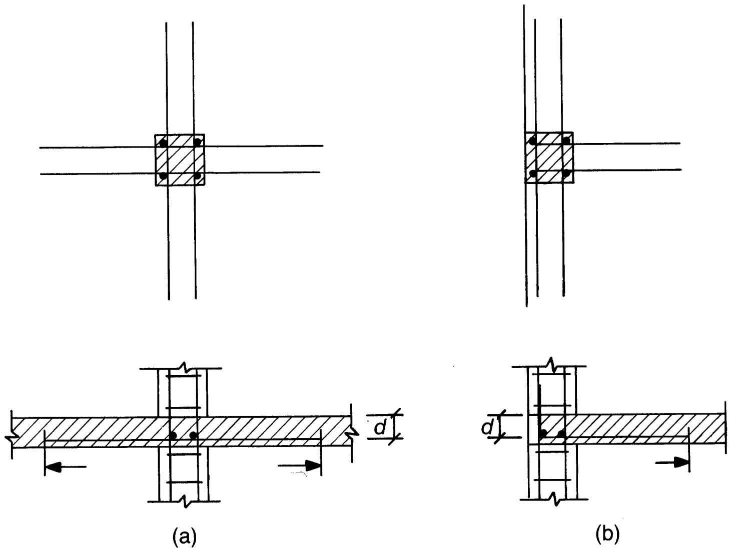


Fig. 6.4.12. Bottom reinforcement at slab-column interfaces: (a) internal column; (b) edge column

In the example A_s is the area of 8 bars for the internal column and 6 bars for the edge column.

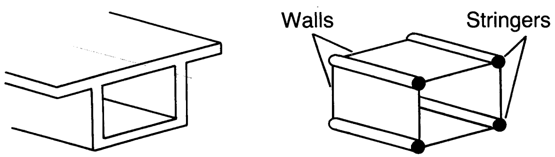


Fig. 6.5.1. Stringer-and-wall model of a box girder

6.4.3.8. Local ductility

To reduce brittleness in the event of a local failure at a slab-column connection the following requirements should be met with respect to bottom steel in the slab crossing the slab-column interfaces.

The total area A_s of reinforcement crossing the interfaces should be such that

$$A_s f_{yd} \geqslant F_{Sd} \tag{6.4–32}$$

On either side of each interface, the steel contributing to A_s should be anchored

(*a*) on the slab side, by a full anchorage length plus a length equal to d
(*b*) on the column side, either by its anchorage in the slab at the other side of the column or by a full anchorage length within the column.

The bars used in A_s should pass inside the main reinforcement of the column and should be of steel type S.

6.5. PLATE ELEMENTS

6.5.1. Scope

This section gives some methods of generalizing the modelling approaches given for specific cases in sections 6.3 and 6.4.

Subsection 6.5.2 treats the design of thin-walled members subjected to action effects M, N, V and T in circumstances where longitudinal shear is not negligible. The method gives a means of determining the in-plane forces per unit width throughout the walls of the section in regions free from discontinuities.

A separate modelling following the principles of section 6.8 is required in regions of discontinuity, e.g. to connect the forces obtained from subsection 6.5.2 to end reactions.

Subsection 6.5.3 gives means of verifying the resistances of plate elements subjected to in-plane loading, while subsection 6.5.4 extends the approach to treat plates subjected to moments as well as in-plane loads.

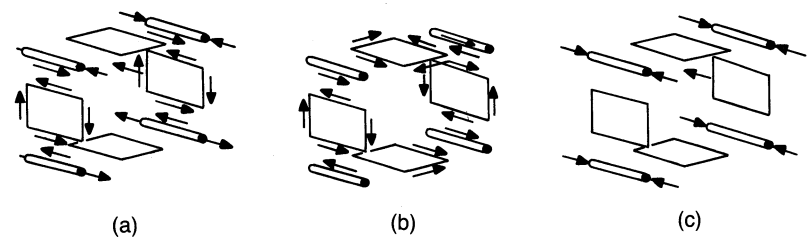


Fig. 6.5.2. Forces on stringer wall + wall model: (a) due to M and V; (b) due to T; (c) due to N

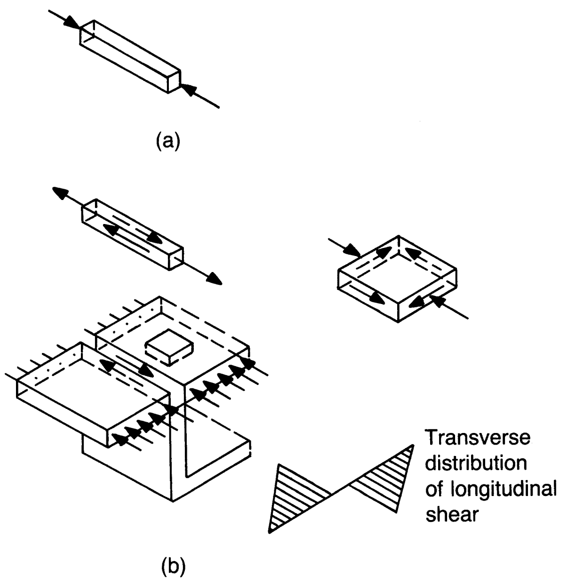


Fig. 6.5.3. Distribution of a stringer force: (a) stage 1; (b) stage 2

6.5.2. Internal forces in thin-walled sections

In the absence of a more accurate analysis the action effects due to M, N, V and T on a part of a member remote from discontinuities may be determined by a two-stage process.

In the first stage the member is modelled by longitudinal 'stringers' situated at the intersections of the walls of the actual member and by webs connecting the stringers.

The stringers are assumed to resist only axial loads and webs are assumed to resist only shear.

The forces in the stringers and webs are determined from the external action effects M, N, V and T. Prestress is treated as an external action with forces $\gamma_p P_{d,o}(x, t)$.

In the second stage forces from the stringers are distributed over the section to produce longitudinal normal forces per unit width and shear forces per unit width. This distribution should not produce any overall effects (M, N, V or T).

The total forces throughout the section can then be obtained as the sums of the forces from the first and second stages.

In the absence of transverse bending effects the member design may then be verified according to subsection 6.5.3.

Where transverse bending is present the verification should be made according to subsection 6.5.4.

6.5.3. Plates subjected to in-plane loading

In-plane loading of a plate may be described in terms of forces per unit width

$$n_{Sdx}, n_{Sdy} \text{ and } v_{Sd}$$

with the axes x and y chosen to coincide with the directions of orthogonal reinforcement.

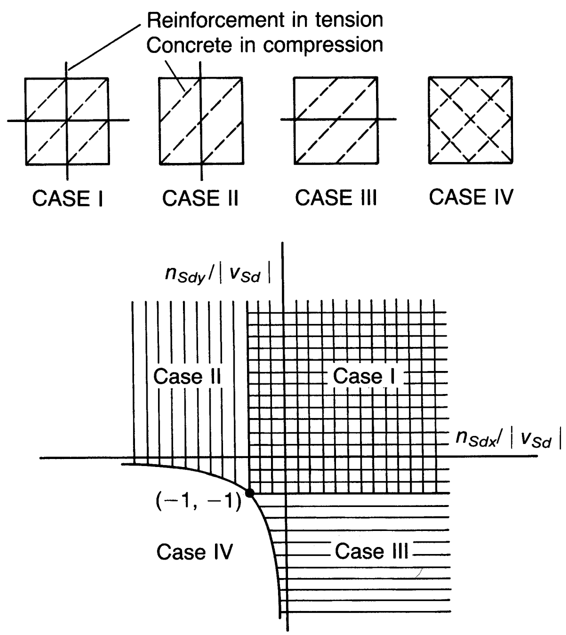


Fig. 6.5.4. Systems of resistance

The internal system providing resistance to in-plane loading may be of four types

- case I tension in reinforcement in two directions and oblique compression in the concrete
- case II tension in reinforcement in the y-direction and oblique compression in the concrete
- case III tension in reinforcement in the x-direction and oblique compression in the concrete
- case IV biaxial compression in the concrete.

The relationship between the applied loading and the systems of resistance is illustrated in Fig. 6.5.4.

The resistances for the ULS are

(*a*) for reinforcement f_{ytd} or $f_{pyd,net}$
(*b*) for concrete in cases I–III f_{cd2}
(*c*) for concrete in case IV f_{cd1} or hf_{cd1} where the coefficient h is the ratio between the biaxial and uniaxial strengths and can be determined from eq. (2.1–11).

θ is the angle between the x-axis and the direction of compression. In cases I–III, the angle θ can be chosen freely so long as the compression is inclined at least 15° to both sets of reinforcement. The minimum local reinforcement is obtained with $\theta = 45°$.

6.5.4. Plates subjected to moments and in-plane loading

The plate may be modelled as comprising three layers. The outer layers provide resistance to the in-plane effects of both the bending and the in-plane loading, while the inner layer provides a shear transfer between the outer layers (see Fig. 6.5.5).

The action effects of the applied loads are expressed as moments and forces per unit width in directions parallel to the orthogonal reinforcement

$$m_{Sdx},\ m_{Sdy},\ m_{Sdxy},\ n_{Sdx},\ n_{Sdy},\ v_{Sd}$$

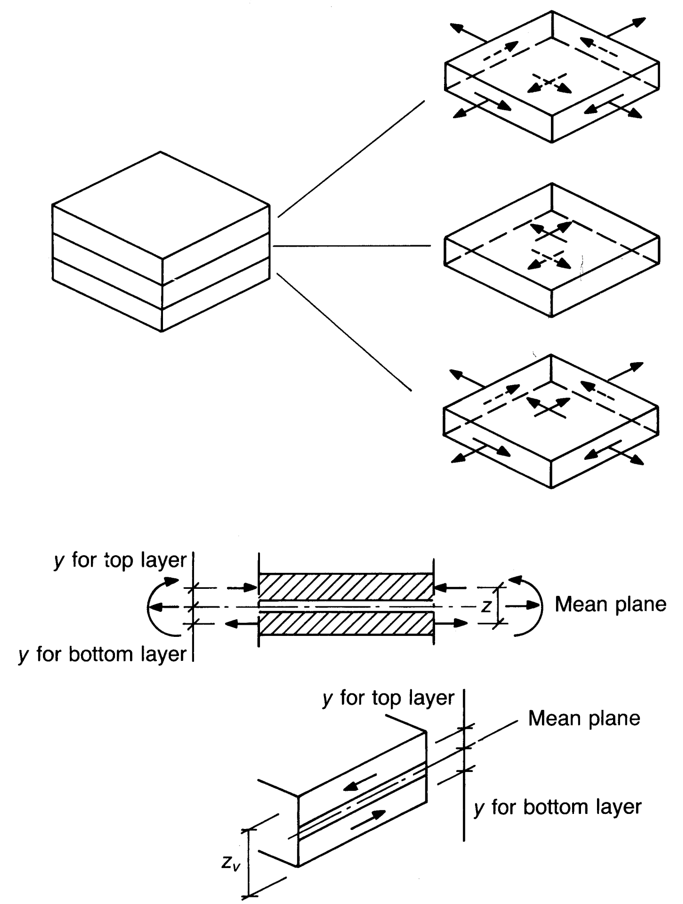


Fig. 6.5.5. Three-layer plate model

An exact determination of z and y values is complex and may require iteration since they depend on the levels of the reinforcement and on the thickness of the concrete layers. A reasonable starting point is to take the z values as $2h/3$ where h is the overall thickness of the plate and to take all values of $(z - y)/z$ as $1/2$.

These produce the following forces per unit width on the plates:

$$n_{pSdx} = n_{Sdx}\frac{(z_x - y)}{z_x} \pm \frac{m_{Sdx}}{z_x}$$

$$n_{pSdy} = \frac{n_{Sdy}(z_y - y)}{z_y} \pm \frac{m_{Sdy}}{z_y}$$

$$v_{pSd} = v_{Sd}\frac{(z_v - y)}{z_v} \pm \frac{m_{Sdxy}}{z_v}$$

where z_x, z_y and z_v are the lever arms between the direct forces in the x and y directions respectively and the shear forces, and y is the distance from the mean plane of the slab to the force in question.

No internal lever arm should be taken greater than the distance between the mean planes of the reinforcement at opposite faces.

The verification of the outer layers can then be made as for plates subjected to in-plane loads (see subsection 6.5.3).

The verification of the inner layer can be made according to subsection 6.4.2.

6.6. ULTIMATE LIMIT STATE OF BUCKLING

6.6.1. Definitions

The ultimate limit state of buckling is defined in clauses 1.6.3.1 and 1.6.3.2.

The ultimate limit state of buckling is in many cases identical with the so-called ultimate limit state of 'first yield' (see clause 5.4.1.2). The design resistance of a member in the ultimate limit state of buckling is therefore in general smaller than the design resistance in the ultimate limit state of an individual cross-section.

A non-linear analysis should in general be used (see clause 5.3.2.1).

6.6.1.1. Range of application

This section applies mainly to uniaxial or mono-dimensional elements made of reinforced or prestressed concrete and subjected to axial compression, with or without bending, for which the effect of combined action of bending and shear as well as bending and torsion may be neglected.

The principles of this section may also be applied to other types of structural elements, such as walls and shells, slender beams subjected to considerable compressive stresses in the compression zone (in which lateral buckling may occur), deep beams and other large or exceptional structures and elements in which significant local deformations may occur or elements made with lightweight aggregate or high strength concrete with normal aggregates or unreinforced concrete.

For the evaluation of shear or torsional deformations, complementary design rules need to be defined (see for example clause 6.6.3.3.4 for lateral buckling of beams).

6.6.1.2. Classification of structures and structural elements

For the purpose of design calculations related to buckling, structures or structural elements may be classified as braced or unbraced depending on the provision of bracing elements or not, and as non-sway or sway, depending on the sensitivity to second order effects due to lateral displacements with respect to the direction of compressive forces.

For the classification sway or non-sway, numerical criteria are given in clause 6.6.3.1.2 for bracing elements and in 6.6.3.1.3 for framed building structures.

The bracing elements considered in this section are elements of a sufficiently high stiffness, with respect to the stiffness of the braced sub-assemblies connected to them, which are able to ensure the stability of the whole structure.

A structure may be considered braced if lateral stability of the structure as a whole is provided by bracing elements designed to resist all lateral forces. It should otherwise be considered as unbraced.

For the classification braced or unbraced, a numerical criterion is given in clause 6.6.3.1.2 for framed building structures.

A framed structure is defined as non-sway if the influence of the displacements of the connections upon the design load effects may be neglected.

Isolated elements are either single elements in compression or integral parts of a structure which may be considered as isolated for design

purposes. In buildings, the cross-section of an isolated element is commonly constant along its length between restraints.

Special structures are large uniaxial structures such as towers, piers, or structures with variable cross-sections in which the longitudinal forces can vary along the axis, or unbraced frames with irregular dimensions and irregular relative stiffness of their elements.

Plane members in compression, such as walls and shells, which may fail by lateral buckling can be verified by the methods presented in this section.

6.6.1.3. Effective length and slenderness

The sensitivity of structures or structural elements to second order effects can be described by the slenderness ratio $\lambda = l_o/i$. The effective length l_o depends on the parameters which influence the deformations and is defined as that length of a pin-ended column which will lead to the same buckling behaviour and load carrying capacity as the real structural element being considered. The radius of gyration i is calculated for the concrete cross-section (gross-section) only and the plane being considered. The actual length of columns, l_{cal}, should be determined according to the rules for the effective span of members (see clause 5.2.3.2).

Slenderness bounds may be defined, especially those below which second order effects may be neglected for the purpose of stability verifications.

Slenderness bounds below which second order effects may generally be neglected should be related to the reduction in bearing capacity, with respect to the ultimate limit state for bending and longitudinal force according to first order theory, of not more than 10%.

In complex structures, where a reliable definition of the slenderness ratio is difficult to obtain, the sensitivity of the structure to second order effects may be expressed in terms of stiffness.

Normally the effective length l_0 is assessed by referring to simplified rules or experience.

For members with variable cross-section, a representative equivalent cross-section may be used in certain cases.

Bearing in mind the uncertainties related to the actual conditions at the end connections of elements, and that all relevant parameters cannot be taken into account, due prudence should be exercised in the determination of the value of l_0, especially for unbraced sway structures.

Attention is drawn to the fact that l_0 depends on the stiffness in the ultimate limit state of the structural elements, which are connected to the element being considered.

The effective length l_0 can in certain cases directly be used for the ultimate limit state design of the structure or structural elements considered as isolated elements.

6.6.2. Requirements

6.6.2.1. General

It has to be verified that

- under the most unfavourable combination of actions
- giving the materials their design strengths and the associated deformation behaviour
- taking into account geometrical imperfections,

the action effects are smaller than or equal to the action effects in the ultimate limit state of buckling.

For further details see clause 1.6.3.2.

6.6.2.2. Differences allowed between rigorous and simplified methods

Simplified or approximate methods should not lead to differences in the design bearing capacity, of the structure or the structural element considered, exceeding 10% in the unsafe direction of that obtained with a rigorous second order analysis.

6.6.2.3. Calculation of deformations

Deformations have to be determined using stress–strain diagrams for concrete which are characterized by at least three parameters which are mutually independent

- the strength f_{cd}
- the strain belonging to the apex ε_{c1}
- the inclination at the origin, which is the tangent modulus of elasticity E_{ci} (refer to clause 2.1.4.2).

The design values f_{cd} and E_{cd} may be determined by dividing the characteristic values f_{ck} and E_{ci} by a safety coefficient $\gamma_c = 1.2$.

Stress–strain diagrams for reinforcing or prestressing steel, as used for cross-section design (ref. clause 2.2.4.3 or 2.3.4.3) should be applied.

The design strength for evaluating the ultimate resistance of critical sections in the 'local' verification is obtained according to subsection 1.4.1 by dividing the characteristic value of the concrete strength by a safety coefficient $\gamma_c = 1.5$, whereas the corresponding values for the overall deformation behaviour are obtained by using a reduced safety coefficient of $\gamma_c = 1.2$, see clause 1.6.3.4.

Tension stiffening effects may be taken into account.

Tension stiffening effects are only significant for small reinforcement ratios and where the ultimate limit state of buckling is reached before yielding of the reinforcement occurs.

Different models are available for taking tension stiffening effects approximately into account. Reduced steel strains ε_s (see subsection 3.2.3), or, alternatively, a constant mean concrete tensile stress $\sigma_{c,t}$ in the effective tension zone around the bars in tension, or variable mean concrete tensile stresses $\sigma_{c,t}(\varepsilon_c)$ in the tension zone of the cross-section may be used.

Creep effects are considered to be significant if the increases of the moments due to second order effects caused by creep and longitudinal action effects exceed 10% of the first order bending moments.

The moment increase instead of the bearing capacity reduction has been chosen because the first criterion allows the analytical determination of corresponding slenderness bounds beyond which creep effects become significant (see eq. (6.6–25)).

Creep effects which increase the deformations in the ultimate limit state and which are likely to reduce the structural stability significantly (refer to clause 6.6.2.1) have to be taken into account. They should be analysed for load combinations according to clauses 1.4.2.2 and 1.6.3.3 defining quasi-

The creep coefficient should be considered as a fundamental basic variable (see section 1.3), where the effects of quasi-permanent combinations are dominant of the total action effects. However, in most practical cases of building structures, the favourable effects of some other parameters are

permanent combinations of loads and applying an appropriate creep coefficient ϕ according to clause 2.1.6.4.3b.

Shrinkage effects can generally be neglected, unless the basic shrinkage of the concrete exceeds the normal range.

neglected and, unless better information on the physical value of the creep coefficient ϕ is given, the notation value ϕ_0 may be used instead of a more unfavourable fractile.

6.6.2.4. Minimum reinforcement ratio

A minimum reinforcement ratio $\omega = (A_s f_{yd})/(A_c f_{cd})$ has to be provided.

The minimumm amount of reinforcement $\min A_s$ may be determined from the design axial force N_{Sd}; $\min A_s = 0.15 N_{Sd}/f_{yd}$.

6.6.3. Design criteria

6.6.3.1. Classification of structures and structural elements

6.6.3.1.1. Isolated elements

Second order effects may be neglected, if the slenderness ratio λ of the column satisfies the criterion

$$\lambda \leqslant \lambda_1 \qquad (6.6\text{–}1)$$

where λ_1 is an appropriate slenderness bound, which takes the decrease of load bearing capacity due to second order effects into account.

First order effects of imperfections need always to be considered for columns or other members and structures for which these imperfections are explicitly defined. Ignoring imperfection effects would result in even greater discontinuity between the column bearing capacities for first order and second order theory.

Slenderness bounds are always considerably affected by the reinforcement ratio. The values given below are lower bounds and are valid for minimum reinforcement. Alternatively, other methods showing the limited influence on the strength reduction may be used.

In the absence of a more rigorous analysis the slenderness bound λ_1 for sway elements may be taken as

$$\lambda_1 = 7.5/\sqrt{\nu_{Sd}} \quad \text{if} \quad \nu_{Sd} \leqslant 0.39 \qquad (6.6\text{–}2)$$

$$\lambda_1 = 12 \quad \text{if} \quad \nu_{Sd} > 0.39 \qquad (6.6\text{–}3)$$

and for non-sway elements

$$\lambda_1 = 7.5(2 - e_{01}/e_{02})/\sqrt{\nu_{Sd}} \quad \text{if} \quad \nu_{Sd} \leqslant 0.39 \qquad (6.6\text{–}4)$$

$$\lambda_1 = 12(2 - e_{01}/e_{02}) \quad \text{if} \quad \nu_{Sd} > 0.39 \qquad (6.6\text{–}5)$$

where

e_{01} denotes the smaller value of the first order eccentricity of the axial action effect at one end of the element considered

e_{02} denotes the greater value

ν_{Sd} denotes the relative design axial force in the bracing elements, $\nu_{Sd} = N_{Sd}/(A_c f_{cd})$.

The introduction of negative values for the ratio e_{01}/e_{02} into eq. (6.6–4) or (6.6–5) is allowed only if the column is designed at least for the combination of the axial force N_{Sd} and a minimum design moment

$$M_{Sd} = N_{Sd}h/20 \qquad (6.6\text{–}6)$$

and the restraints at the column ends are able to resist this moment, so that a double curvature deflection develops.

For the definition of braced structures see clause 6.6.1.2.

Braced structures comprise two groups of elements with different stiffnesses against horizontal actions.

The arrangement of bracing elements should be such that they can also resist torsional actions.

This clause, which refers to the frequent combination, is intended to limit cracking, not to avoid it totally.

If a more precise model is not available, the following formula can be applied for the calcualtion of λ_b in the case of regular braced frames with approximately equal height and approximately equal vertical loads in each storey

$$\lambda_b = \frac{h_{tot}\sqrt{(F_v/E_{cm}I_c)}}{0.01\beta\sqrt{v_{Sd}}} \qquad (6.6\text{–}8)$$

6.6.3.1.2. Braced structure

Braced structures may be analysed by considering first the braced sub-assembly neglecting any horizontal load and assuming horizontal restraints at each storey level. Its individual columns may be designed as non-sway isolated elements. Thereafter the bracing elements will be designed as cantilever columns submitted to all horizontal loads acting on the structure and to the reactions of the sub-assembly.

A building structure can be considered as braced if its bracing elements are reasonably symmetrically distributed within the building and where it can be shown, by means of a first order linear analysis with member stiffnesses corresponding to uncracked cross-sections, that they are able to attract, at the foundations level, a shear force at least equal to 90% of the sum of the horizontal forces acting on the building. In addition, the bracing elements have to remain uncracked in service conditions under the frequent combination of all horizontal loads and the corresponding vertical loads. The latter criterion may be considered to be met if the concrete tensile stress σ_{ct} does not exceed f_{ctm} defined in clause 2.1.3.3.1.

Depending on their slenderness ratios the bracing elements may be considered as non-sway or sway and designed accordingly as short columns or slender columns, i.e. according to the first or second order theory. The corresponding criterion for non-sway is

$$\lambda_b \leqslant \lambda_1 \qquad (6.6\text{–}7)$$

where

- λ_b is the equivalent slenderness ratio of the bracing elements taking into account the second order moments due to the loads applied to the braced sub-assembly
- λ_1 is the slenderness bound for isolated elements according to clause 6.6.3.1.1.

If this criterion is fulfilled, the bracing elements may be designed according to a first order analysis taking into account the effects of imperfections.

with

$$\beta = 1.8\text{–}1.44/(n + 0.8) \qquad (6.6\text{–}9)$$

where

n denotes the number of storeys
h_{tot} denotes the total height of the structure measured from the top surface of the foundation or from a non-deformable sub-stratum
$E_{cm} I_c$ denotes the sum of the nominal flexural stiffnesses of all the vertical bracing elements acting in the direction under consideration
F_v denotes the sum of all vertical loads in service conditions (rare combinations), acting on the bracing elements and on the braced sub-assembly, with $\gamma_F = 1.0$ (the effect of $\gamma_F > 1.0$ is covered by the factor 0.01 in eq. (6.6–8)).
v_{Sd} denotes the relative design axial force in the bracing elements, $v_{Sd} = N_{Sd}/(A_c f_{cd})$.

If the stiffness of the bracing elements is variable along the height, an equivalent stiffness should be used. As an approximation, the equivalent stiffness may be derived from the assumption that it leads to the same horizontal deflection under a unit load as the actual stiffnesses of the structure.

The limitation of the equivalent slenderness for bracing elements is related to the 10% decrease in bearing capacity under which structures can be considered non-sway (see clause 6.6.3.1.3).

As the actual load effects in the braced sub-assembly are likely to be different from the design values, it is recommended that the members of the sub-assembly have adequate ductility.

6.6.3.1.3. Non-sway framed structures

Non-sway structures may be analysed according to the first order theory. This analysis can be done with nominal or with reduced member stiffnesses.

A building frame can be considered as non-sway if the displacements of the connections would only result in a 10% increase of the relevant first order bending moments. This analysis should be made taking into account either the non-linear behaviour of the material or, as a simplification, adequately reduced member stiffnesses.

Individual non-sway compression members should be considered as isolated elements and be designed accordingly.

In general, for sway structures a second order analysis will be necessary.

Simplified methods, introducing, for example, instead of the effects of geometrical imperfections increased horizontal design loads or nominal bending moments which take account of second order effects, may be used.

For economic reasons, the second order effects should always be limited. In this case, a rough assessment of their effects may be adequate. A simplified procedure is described here below.

The deflected shape of the frame is assumed to be straight with an angle of α'' from the vertical so that the second order effects may be expressed by

the action of additional horizontal forces $\Delta H_{Sd} = \alpha'' V_{Sd}$. Assuming uniformly distributed horizontal and vertical loads H_{Sd} and V_{Sd}, the deformation $\Delta a'$ in the plane of the frame due to these additional forces ΔH_{sd} can be determined by multiplying the first order deflection $a'(H_{Sd})$ due to forces H_{Sd} by the ratio $\alpha''\Sigma(V_{Sd}x)/\Sigma(H_{Sd}x)$. Finally, the total slope α'' can then be calculated from the first order top deflection of the frame $a' = a'(H_{Sd}) + a'(V_{Sd})$ and taking also the deviation α_a from the vertical according to eq. (6.6–13) into account.

In the following x denotes the vertical co-ordinate of the application point of the relevant horizontal load H_{Sd} and the vertical load V_{Sd} (see Fig. 6.6.1).

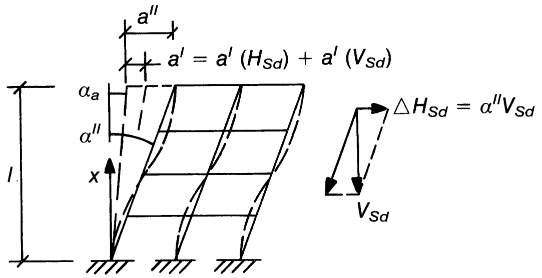


Fig. 6.6.1. Simplified calculation of the maximum displacement a'' *due to second order effects*

$$\alpha'' = \alpha_a + a'/l + \Delta a'(\Delta H_{Sd})/l \qquad (6.6\text{–}10)$$

$$= \alpha_a + a'/l + \frac{\Sigma(\Delta H_{Sd}x)}{\Sigma(H_{Sd}x)}\frac{a'(H_{Sd})}{l}$$

$$= \alpha_a + a'/l + \frac{\alpha''\Sigma(V_{Sd}x)}{\Sigma(H_{Sd}x)}\frac{a'(H_{Sd})}{l}$$

$$\alpha'' = \frac{\alpha_a + a'/l}{1 - (\Sigma V_{Sd}x/\Sigma H_{Sd}x)(a'(H_{Sd})/l)} \qquad (6.6\text{–}11)$$

The first order deflections a' for design actions H_{Sd} and V_{Sd} should be calculated by considering the stiffness reduction due to cracking.

In many cases, a rough estimate is adequate. In the case where the resulting increase of the horizontal loads is not greater than 25%, the deflection may be assumed to be twice the deflection obtained by a first

order linear frame analysis with the stiffness $E_{cm}I_c$. This is equivalent to the assumption of a 50% reduction in stiffness. Otherwise, it has to be shown that the assumed stiffness reduction, which may be different for the different members, corresponds reasonably well with the resulting state of stress. The whole structure should be designed for increased horizontal loads $H_{Sd,ef}$

$$H_{Sd,ef} = H_{Sd} + \alpha'' V_{Sd} = (1 + \alpha'' V_{sd}/H_{Sd})H_{Sd} \qquad (6.6\text{–}12)$$

The magnification factor $(1 + \alpha'' V_{Sd}/H_{Sd})$ for the horizontal loads constitutes a good indication of the sensitivity of the structure to sway. If the factor is large, it may be advisable to modify the design.

The simple procedure is a special form of the P/Δ-method.

The design value of geometrical imperfections should not include effects of other structural imperfections, such as residual stresses by shrinkage and creep, inhomogeneities or locally poor concrete. These structural imperfections should be covered by adequate design values for the stress–strain diagram of concrete and/or by using appropriately chosen safety coefficients γ_c. However, the design value should cover the secondary effects of actions (e.g. thermal gradients) which are not considered separately; such effects may be important for high rise buildings and very slender structures.

This rule is mainly intended for common framed building structures. m is the total number of elements, whether they are bracing or not, that support vertical loads and are horizontally tied together. In the case shown in Fig. 6.6.2 $m = 2$ (one bracing element in A and one non-bracing element in B).

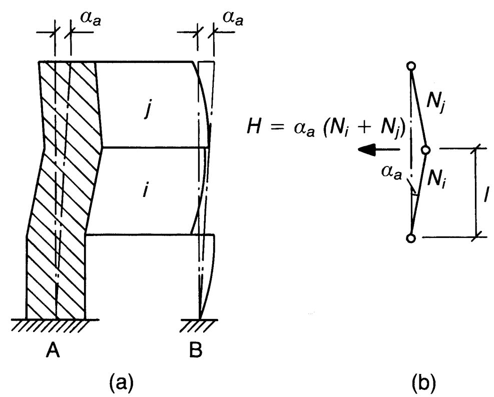


Fig. 6.6.2. Representation of geometrical imperfections by a deviation α_a from the vertical

6.6.3.2. Imperfections

Geometrical imperfections should be taken into account.

The effects of geometrical imperfections may be taken into account in different ways depending on the type of element considered.

(*a*) For multi-storey structures, a deviation α_a of the structure from the vertical may be assumed as shown, for example, in Fig. 6.6.2(a) for a braced frame

$$\alpha_a = 1/(100\sqrt{l}) \leqslant 1/200 \qquad (6.6\text{–}13)$$

where l denotes the member length (m).

In the case where m vertical multistorey continuous elements are horizontally tied together, α_a as defined by eq. (6.6–13) may be reduced by a factor α_m obtained from eq. (6.6–14)

$$\alpha_m = \sqrt{[0.5(1 + 1/m)]} \qquad (6.6\text{–}14)$$

Figure 6.6.2(b) gives guidance on how to calculate the local horizontal forces H in the floors transferring the stabilizing forces from the braced members to the bracing elements. These local forces H, determined with the local length of the braced members, need not be summed for verifying the bracing element. The bracing element has to be verified for horizontal forces corresponding to the inclination as derived from the length of the bracing element.

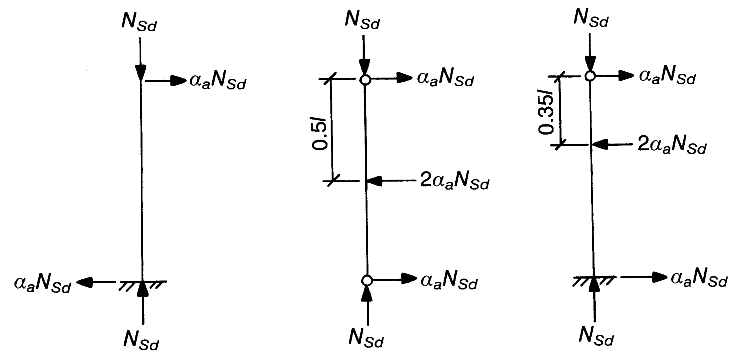


Fig. 6.6.3. Transverse forces for considering effects of geometrical imperfections

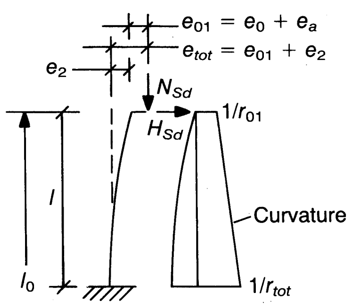


Fig. 6.6.4. Model column

(*b*) For isolated elements, the effects of geometrical imperfections may be taken into account by increasing the eccentricity of the design normal forces by an additional eccentricity e_a acting in the most unfavourable direction and defined by eq. (6.6–15):

$$e_a = \alpha_a l/2 \tag{6.6–15}$$

Alternatively the effects of geometrical imperfections can be taken into account by additional forces equal to α_a times the normal forces, N_{Sd}, and acting transversely to them in the most unfavourable direction. In consequence, pin-ended columns may be loaded according to Fig. 6.6.3 by a transverse load at mid-span which is $2\alpha_a$ times the normal force.

For columns in sway frames, a displacement from the vertical may be considered in order to account for geometrical imperfections or, alternatively, modified horizontal loads at the ends equivalent to this displacement may be used.

6.6.3.3. Simplified design methods for isolated elements

6.6.3.3.1. Second order deformation

A design method may be used which adopts a simplified shape for the deformed axis of the member. The second order deflection e_2 is then calculated as a function of the member length l, the eccentricities e_{01} and e_{02} of the axial force at the ends of the member and the curvature $1/r_{tot}$ in the critical section with the total eccentricity e_{tot} according to eq. (6.6–16):

$$e_{tot} = e_0 + e_a + e_2 \tag{6.6–16}$$

where

e_0 denotes the first order eccentricity $e_0 = M_{Sd,1}/N_{Sd}$

$M_{Sd,1}$ denotes the maximum design bending moment
N_{Sd} denotes the applied design axial force

The 'model column' as shown in Fig. 6.6.4 is a cantilever column with constant cross-section, fixed at the base and free at the top. It is being bent in single curvature under loads and moments which give the maximum moment at the base.

In the case of constant reinforcement the maximum deflection may be assumed to be

$$e_2 = 0.1K_1 l^2(4/r_{tot} + 1/r_{01}) \quad (6.6\text{–}17)$$

$$= 0.1K_1 l^2(4 + e_{01}/e_{tot})1/r_{tot} \quad (6.6\text{–}18)$$

and when the reinforcement is curtailed in accordance with the bending moment diagram, it may be assumed to be

$$e_2 = 0.5K_1 l^2/r_{tot} \quad (6.6\text{–}19)$$

where

$1/r_{tot}$ denotes the curvature associated with the eccentricity e_{tot}

$1/r_{01}$ denotes the curvature associated with the eccentricity e_{01} and which may be assumed to be $(e_{01}/e_{tot})1/r_{tot}$

K_1 denotes a coefficient which is introduced in order to avoid discontinuity of the function describing the design bearing capacity when the slenderness bound λ_1 is exceeded; it is obtained from eq. (6.6–20)

$$K_1 = 2(\lambda/\lambda_1 - 1) \quad \text{for} \quad \lambda_1 \leqslant \lambda \leqslant 1.5\lambda_1 \quad (6.6\text{–}20)$$

$$K_1 = 1 \quad \text{for} \quad \lambda > 1.5\lambda_1 \quad (6.6\text{–}21)$$

The curvature $1/r_{tot}$ is derived from the equilibrium of the internal and external forces.

For pin-ended columns with constant cross-section and reinforcement and subjected to first order moments varying linearly along their length and where $|e_{02}| > |e_{01}|$, an equivalent eccentricity e_e may be taken as

$$e_e = 0.6e_{02} + 0.4e_{01} \quad (6.6\text{–}22)$$

The stability verification may then be done as for a 'model column' but having only half the length l of the real column. The cross-section design with e_{02} is also necessary.

A fictitious curvature $1/r_{tot}$ in eqs. (6.6–17, –18 and –19) may be derived for rectangular cross-sections with symmetrically arranged reinforcement in a top and bottom layer from

e_a denotes the additional eccentricity according to eq. (6.6–15)

e_2 denotes the eccentricity due to second order deflection.

In cases where great accuracy is not required, the verification of stability may be done by checking the critical section for the ultimate limit state of bending and axial force under the axial load N_{Sd} and a fictitious design value of the second order deflection e_2.

$$1/r_{tot} = 2K_2\varepsilon_{yd}/z_s \tag{6.6–23}$$

where

$\varepsilon_{yd} = f_{yd}/E_s$ is the design yield strain of steel reinforcement

z_s is the distance between compression and tension reinforcement, approximately $z_s = 0.9d$

K_2 is a coefficient, taking into account the decrease of the curvature with increasing axial force as defined by eq. (6.6–24)

$$K_2 = (N_{ud} - N_{Sd})/(N_{ud} - N_{bal}) \leqslant 1 \tag{6.6–24}$$

where

N_{ud} is the design ultimate capacity of the section subjected to axial load only, it may be taken as $0.85f_{cd}A_c + f_{yd}A_s$

N_{Sd} is the actual design axial force

N_{bal} is the design axial load which, when applied to the concrete section, maximizes its ultimate moment capacity; for symmetrically reinforced rectangular sections it may be taken as $0.4f_{cd}A_c$.

It will always be conservative to assume $K_2 = 1$.

For columns with cross-sections other than rectangular or with distributed reinforcement equivalent values may be used for z_s.

One of the two following simplified procedures may be used to treat the creep effects approximately.

The element is directly calculated for the ultimate limit state applying the most unfavourable combination of factored actions and including the effects of geometrical imperfections, by using a modified stress–strain relationship, which is obtained by magnifying all strains with the factor $K_3 = 1 + \alpha\beta$. When using eq. (6.6–23) for determining the fictitious curvature $1/r_{tot}$, the curvature may be multiplied by $0.5K_3$ to take creep effects into account.

The creep effects may also be introduced as an additional eccentricity e_c according to eq. (6.6–28)

$$e_c = (e_0 + e_a)\left[\exp\left(\frac{\phi}{N_E/N_{Sg} - 1}\right) - 1\right] \tag{6.6–28}$$

where

N_{Sg} denotes the axial force in the element, under the quasi-permanent combination of actions

6.6.3.3.2. Creep effects

Creep effects may be neglected when at least two of the following conditions are fulfilled simultaneously

$$\lambda \leqslant 53/(\sqrt{(\nu_c)}f_{ck}^{1/3}) \quad (f_{ck} \text{ in (MPa)}) \tag{6.6–25}$$

$$e_0 \geqslant 2h \tag{6.6–26}$$

$$\alpha\beta \leqslant 0.2 \tag{6.6–27}$$

where

$\nu_c = N_{Sg}/(f_{ck}A_c)$ and where N_{Sg} denotes the axial force under quasi-permanent actions

e_0 denotes the first order eccentricity of N_{Sg}

h denotes the height of the cross-section

α denotes the ratio of the design bending moment N_{Sg} under the quasi-permanent actions and the total factored design normal force N_{Sd} considered for the ultimate limit state

β denotes the ratio of the quasi-permanent design bending moment M_{Sg} standard and the total factored design bending moment M_{Sd} considered for the ultimate limit state.

$N_E = E_{cm} I_c (\pi / l_0)^2$ denotes the critical Euler-load of the element, I_c is the moment of inertia of the uncracked concrete section; if cracking is likely to occur under permanent actions an appropriate reduction of I_c should be used

e_c is the remaining additional eccentricity due to creep effects when the column is fictitiously considered as unloaded.

Out-of-plane buckling means buckling transverse to the loaded plane in the case of mono-axial eccentricity.

Geometrical imperfections according to eq. (6.6–15) have to be taken into account by additional eccentricities e_{ay} and e_{az} for both principal planes and by applying them simultaneously.

$$e_{y1} = e_{0y} + e_{ay}$$

$$e_{z1} = e_{0z} + e_{az}$$

Both additional eccentricites are equal, e.g. $e_{ay} = e_{az}$, if the actual lengths between restraints in both principal planes are equal, e.g. $l_y = l_z$.

6.6.3.3.3. Biaxial eccentricities and out-of-plane buckling

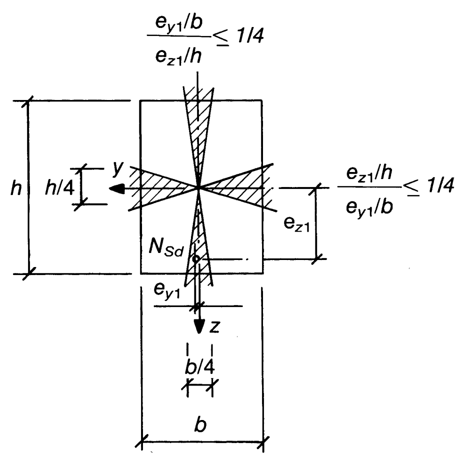
For members with rectangular cross-sections, separate verifications in the two principal planes y and z are permissible, if the point of application of N_{Sd} is located close to one principal axis, e.g. within the hatched zones in Fig. 6.6.5. The ratios of the corresponding eccentricities e_{y1}/b and e_{z1}/h have to satisfy one of the following conditions: either

$$(e_{z1}/h)/(e_{y1}/b) \leqslant 1/4 \qquad (6.6\text{–}29)$$

or

$$(e_{y1}/b)/(e_{z1}/h) \leqslant 1/4 \qquad (6.6\text{–}30)$$

The eccentricities e_{y1} and e_{z1} are those in directions of the section dimensions b and h respectively and including imperfections e_a as defined in eq. (6.6–15). A rigorous analysis is required if the above conditions are not met.



For a separate verification with the reduced width h all the reinforcement may be taken into account, also the reinforcement outside the compression zone.

The basic idea of this simplified method is to show the existence of equilibrium between actions and member resistances for the deformed system. For this purpose, the structural system, loading and cross-sections as well as the deformation conditions have to be known. The system, loading and cross-section are usually known or determined by the first order design for bending. The actual deformation conditions, which imply the lateral deflection and the rotation related to the shear centre M as indicated in Fig. 6.6.7, are not known. Therefore, the deformation curve is commonly approximated by an assumed function and the magnitude of the deflection is fixed using an ultimate limit state assumption for the admissible rotation θ_{adm} of the cross-section.

The following developments of the commentary are intended for reinforced concrete beams, as well as for prestressed concrete beams. The symbols y and z denote the horizontal and vertical directions, respectively.

For the limitation of θ_{adm} and the final verification two conditions have to be considered.

(*a*) *Biaxial bending*

$$M_{z1d} = M_{yd}(\theta + \theta_a)$$

$$\theta_{B,adm} = M_{z1,d}/M_{yd} - \theta_a \qquad (6.6\text{–}32)$$

Where $e_{1z}/h > 0.2$, separate verification is permissible only if the check for bending in the y-direction is based on the reduced width h', as shown in Fig. 6.6.6. The value h' may be determined as the height of the compression zone and on the assumption of linear stress distribution and uncracked concrete.

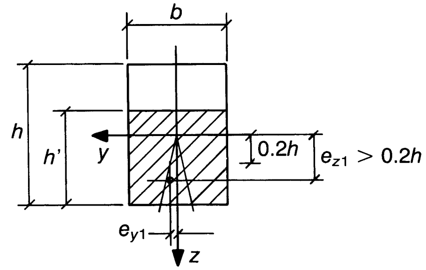


Fig. 6.6.6. Reduced effective width h′ *where* $e_{z1} > 0.2h$

6.6.3.3.4. Lateral buckling of beams

For the verification of adequate safety against lateral buckling of slender beams with slender chords, a second order analysis may be used, which adopts a simplified shape for the lateral deflection and a simplified expression for the rotation of the mid-section caused by torsion. The mid-section of the beam has to be designed for biaxial bending according to the mid-section rotation. The end restraints (in general acting as a fork) shall resist the resulting second order torsion effects caused by the lateral deflection.

Favourable influences such as bracing by horizontal trusses or torsional restraints from purlins may be considered.

The effects of geometrical imperfections may be taken into account by a lateral pre-deflection e_a equal to the eccentricity according to eq. (6.6–15) together with an additional rotation of the mid-section of

$$\theta_a = 1/200 \qquad (6.6\text{–}31)$$

The effects of creep may be taken into account approximately by doubling both values of geometrical imperfections.

The torsional stiffness GI_T may be considered as constant along the beam and corresponding to the depth of the compression zone under first order bending moment M_{yd} with its maximum value neglecting the effects of geometrical imperfections. The shear modulus may be taken as $G = 0.4E_{cd}$.

(*b*) *Onset of cracking due to torsion near the support*
Assuming that no longitudinal stresses result from bending, the principal tensile stresses ae identical to the torsional stresses. The admissible rotation can then be determined from the torsional cracking moment T_r. For a sinusoidal shape of the acting second order torsional moments this admissible rotation is defined by

$$\theta_{T,adm} = \frac{T_r l}{\pi G I_T} \tag{6.6–33}$$

where

$$T_r = 0.25 f_{ck}^{2/3} W_t \tag{6.6–34}$$

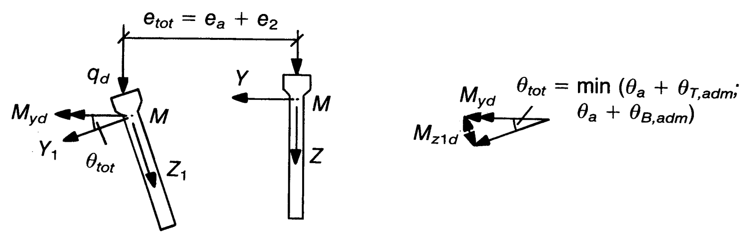


Fig. 6.6.7. Deformed position of the cross-section in the middle of the beam

In most cases $\theta_{T,adm}$ is smaller than $\theta_{B,adm}$ and therefore decisive for the design.

The admissible lateral deflection $e_{2,adm}$ has to be limited, so that the second order torsional moment T_d does not exceed T_r. For constant vertical loading q_d along the beam, $q_d l = Q_d$, it can approximately be assumed

$$T_d = T_r = Q_d(e_{2,adm} + e_a)/\pi \tag{6.6–35}$$

$$e_{2,adm} = \pi T_r / Q_d - e_a \tag{6.6–36}$$

The beam has to be designed such, that the lateral deflection

$$e_2 = \frac{Q_d \theta_{tot} l^3}{F.I} 0.0112 \tag{6.6–37}$$

does not exceed the admissible lateral deflection $e_{2,adm}$. The lateral bending stiffness EJ_z can be determined using only the compressive zone due to the first order bending moment M_{yd} and the reinforcement.

The given fatigue strength for concrete is valid for concrete tested under sealed conditions (see subsection 2.1.7). The fatigue strength for steel is given as well for normal environment as for marine environment.

Notation

D is fatigue damage

n_{Si} is the number of acting stress cycles associated with the stress range for steel, and the actual stress levels for concrete

N_{Ri} or N is the number of resisting stress cycles

n is the foreseen number of cycles in the desired design lifetime

$\Delta\sigma_{Ss}$ is the steel stress range under the acting loads

$\Delta\sigma_{Rsk}(n)$ is the stress range relevant to n cycles obtained from a characteristic fatigue strength function

$S_{cd,\max}$ is the maximum compressive stress levels

$S_{cd,\min}$ is the minimum compressive stress levels

ΔS_{cd} is the stress range

$S_{td,\max}$ is the maximum tensile stress level

$\sigma_{c,\max}$ is the maximum compressive stress

$\sigma_{c,\min}$ is the minimum compressive stress

$\sigma_{ct,\max}$ is the maximum tensile stress

η_s is the factor which increases the stress in the reinforcing steel due to differences in bond behaviour of prestressing and reinforcing steel

η_c is the averaging factor of concrete stresses in the compression zone considering the stress gradient

$f_{cd,fat}$ is the design fatigue reference strength for concrete under compression

$f_{ctd,fat}$ is the design fatigue reference strength for concrete under tension

θ_{fat} is the angle between the web compression and the chords valid for verification of the reinforcement.

6.7. ULTIMATE LIMIT STATE OF FATIGUE

6.7.1. Scope

The following design rules apply for the entire lifetime of concrete. The rules for reinforcing and prestressing steel apply for more than 10^4 repetitions; low-cycle fatigue is not covered.

The verification of the design principle (see clause 1.6.4.2) can be performed according to the three methods given in subsections 6.7.3, 6.7.4 and 6.7.5, with an increasing refinement. The models for the analysis of stresses in reinforced and prestressed concrete members under fatigue loading are treated in subsection 6.7.2 as well as concrete stress gradients.

Subsection 6.7.6 deals with shear design and in 6.7.7 a method for calculating the increased deflections under fatigue loading is given. The relevant combination of loads is treated in clause 1.6.4.5.

6.7.2. Analysis of stresses in reinforced and prestressed members under fatigue loading

Linear elastic models may generally be used, and reinforced concrete in tension is considered to be cracked. The ratio of moduli of elasticity for steel and concrete may be taken as $\alpha = 10$.

In the case of prestressed members it should be verified if the relevant section is sensitive to cracking. This holds true if any combination of loads (see clause 1.6.6.5) causes tensile stresses in the surface fibre and then the stress ranges for reinforcing steel and prestressing steel should be calculated as though the member is in the cracked state.

The effect of differences in bond behaviour of prestressing and reinforcing steel has to be taken into account for the stresses in the reinforcing steel. Unless a more refined method is used, this can be done using a linear elastic model for stress calculation which fulfils the compatibility in strains and multiplying the stress in the reinforcing steel by the following factor:

$$\eta_s = \frac{1 + (A_p/A_s)}{1 + (A_p/A_s)\sqrt{[\zeta(\phi_s/\phi_p)]}} \geqslant 1 \qquad (6.7\text{–}1)$$

where

A_s is the area of reinforcing steel
A_p is the area of prestressing steel
ϕ_s is the smallest diameter of reinforcing steel in the relevant section
ϕ_p is the diameter of prestressing steel (for bundles an equivalent diameter has to be chosen $1.6\sqrt{A_p}$ where A_p is the cross-section area of the bundle)
ζ is the ratio of bond strength of prestressing steel and high-bond reinforcing steel.

Post-tensioned members

$\zeta = 0.2$ for smooth prestressing steel
$\zeta = 0.4$ for strands
$\zeta = 0.6$ for ribbed prestressing wires
$\zeta = 1.0$ for ribbed prestressing bars

Pretensioned members

$\zeta = 0.6$ for strands
$\zeta = 0.8$ for ribbed prestressing steels

The stress gradient for concrete in the compression zone of a cracked section may be taken into account by multiplying the maximum stress in the

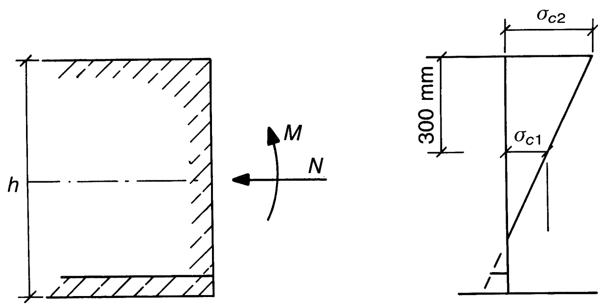


Fig. 6.7.1. Definition of stresses σ_{c1}, σ_{c2}

$$\eta_c = \frac{1}{1.5 - 0.5|\sigma_{c1}|/|\sigma_{c2}|} \tag{6.7–2}$$

where

$|\sigma_{c1}|$ is the lower absolute value of the compressive stress within a distance no more than 300 mm from the surface under the relevant load combination

$|\sigma_{c2}|$ is the larger absolute value of the compressive stress within a distance no more than 300 mm from the surface under the same load combination as for $|\sigma_{c1}|$.

6.7.3. Verification by the simplified procedure

This procedure is only applicable to structures subjected to a limited number ($\leqslant 10^8$) of low stress cycles.

Steel

The fatigue requirements will be met, if the maximum calculated stress range under the frequent combination of loads, max $\Delta\sigma_{Ss}$, satisfies

$$\gamma_{Sd} \max \Delta\sigma_{Ss} \leqslant \Delta\sigma_{Rsk}/\gamma_{s,fat} \tag{6.7–3}$$

where $\Delta\sigma_{Rsk}$ is the characteristic fatigue strength at 10^8 cycles. Values for $\Delta\sigma_{Rsk}$ are given in Tables 6.7.1 and 6.7.2.

Values for $\gamma_{s,fat}$ and $\gamma_{c,fat}$ are given in clause 1.6.4.4.

Concrete

Detailed fatigue design need not be carried out if the maximum calculated stress under the frequent combination of loads, $\sigma_{c,\max}$ (compression), $\sigma_{ct,\max}$ (tension), respectively, satisfy the following

The fatigue reference strength is defined as follows (see also clause 2.1.7.1).

Compression

$$f_{cd,\mathrm{fat}} = 0.85\beta_{cc}(t)\left[f_{ck}\left(1 - \frac{f_{ck}}{25 f_{cko}}\right)\right]\Big/\gamma_c$$

where

$\beta_{cc}(t)$ is the coefficient which depends on the age of concrete t in days when fatigue loading starts (see clause 2.1.6.1)

f_{cko} = 10 MPa (reference strength).

Compression:

$$\gamma_{Sd}\sigma_{c,\max}\eta_c \leqslant 0.45 f_{cd,fat} \tag{6.7–4}$$

where η_c is the averaging factor considering the stress gradient eq. (6.7–2).

Tension

$$\gamma_{Sd}\sigma_{ct,\max} \leqslant 0.33 f_{ctd,fat} \tag{6.7–5}$$

Tension

$$f_{ctd,fat} = f_{ctk0.05}/\gamma_{c,fat}$$

For value of $\gamma_{c,fat}$ see clause 1.6.4.4.
For $\sigma_{c,\max}$, $\sigma_{ct,\max}$ see clause 1.6.4.2.

When the unique value Q can be chosen satisfactorily, (e.g. as fatigue equivalent) this method is a more precise assessment than the simplified procedure.

When it is considered necessary to carry out fatigue tests to determine the performance of reinforcing steel, the tests should be made according to RILEM-FIP-CEB Recommendations, 1973. These data should normally be expressed as 5% fractiles and 75% confidence levels.

The characteristic fatigue strength function for steel consists of segments (see Fig. 6.7.2) of the form $\Delta\sigma^m_{Rsk} N =$ const. Values for the S–N curves are given in Tables 6.7.1 and 6.7.2.

The values given in Tables 6.7.1 and 6.7.2 are characteristic and do not incorporate partial safety factors. These values or higher values shall be validated by appropriate approval documents.

The code does not cover coiled and restraightened bars.

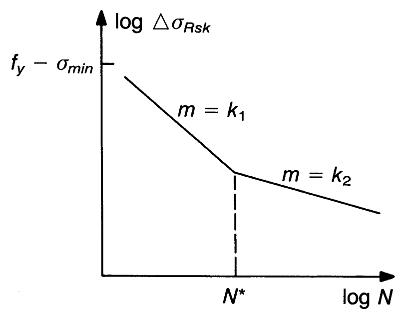


Fig. 6.7.2. Shape of the characteristic fatigue strength curves (S–N curves) for steel

6.7.4. Verification by means of single load level

This method takes account of the required lifetime with a foreseen number, n, of cycles. This number intervenes in the verification with the maximum fatigue effects of the action, Q, as defined in clauses 1.6.4.2.(c), 6.7.2 and below.

Steel

The fatigue requirement will be met if the calculated maximum acting stress range, max $\Delta\sigma_{Ss}$, satisfies

$$\gamma_{Sd} \max \Delta\sigma_{Ss} \leqslant \Delta\sigma_{Rsk}(n)/\gamma_{s,fat} \qquad (6.8\text{–}6)$$

where

n is the foreseen number of cycles in the required design lifetime,
$\Delta\sigma_{Rsk}(n)$ is the stress range relevant to n cycles obtained from a characteristic fatigue strength function.

Table 6.7.1. Parameters of S–N curves for reinforcing steel (embedded in concrete)

	N^*	Stress exponent		$\Delta\sigma_{Rsk}$ (MPa)[e]	
		k_1	k_2	At N^* cycles	At 10^8 cycles
Straight and bent bars $D \geqslant 25\phi$					
$\phi \leqslant 16$ mm	10^6	5	9	210	125
$\phi > 16$ mm[a]	10^6	5	9	160	95
Bent bars $D < 25\phi$[b]	10^6	5	9	–[c]	–[c]
Welded bars[b] including tack welding and butt joints mechanical connectors	10^7	3	5	50	30
Marine environment[b],[d]	10^7	3	5	65	40

In case appropriate information is provided by specific approval documents for the steel to be used, higher fatigue strength values may be used accordingly.

[a]The values given in this line represent the S–N curve of a 40 mm bar; for diameters between 16 and 40 mm interpolation between the values of this line and those of the line above is permitted.
[b]Most of these S–N curves intersect the curve of the corresponding straight bar. In such cases the fatigue strength of the straight bar is valid for cycle numbers less than that of the intersection point.
[c]Values are those of the according straight bar multiplied by a reduction factor ξ depending on the ratio of the diameter of mandrel D and bar diameter ϕ: $\xi = 0.35 + 0.026D/\phi$.
[d]Valid for all ratios D/ϕ and all diameters ϕ.
[e]In cases where $\Delta\sigma_{Rsk}$-values calculated from the S–N curve exceed the stress range $f_{yd} - \sigma_{min}$, the value $f_y - \sigma_{min}$ is valid.

The values given in Table 6.7.2 are on the safe side compared to the strength values for the basic material given in section 2.3.

Table 6.7.2. Parameters of S–N curves for prestressing steel (embedded in concrete)

Prestressing steel	N^*	Stress exponent		$\Delta\sigma_{Rsk}$ (MPa)	
		k_1	k_2	At N^* cycles	At 10^8 cycles
Pretensioning					
Straight steels	10^6	5	9	160	95
Post-tensioning					
Curved tendons[a]	10^6	3	7	120	65
Straight tendons	10^6	5	9	160	95
Mechanical connectors	10^6	3	5	80	30

[a]In cases where the S–N curve intersects that of the straight bar, the fatigue strength of the straight bar is valid.

The reduction of the $\Delta\sigma_{Rsk}$ values of curved tendons compared with the values of straight tendons is due to fretting corrosion which results from the lateral pressure and slip between prestressing strands and/or ribs of the steel sheaths.

Concrete

The fatigue requirements under cyclic loading will be met if the required lifetime (number of cycles) is less than or equal to the number of cycles to failure:

$$n \leqslant N$$

N should be calculated from the fatigue strength functions given below.

Characteristic S–N curves for concrete can be used without any restriction for frequencies higher than 0.1 Hz. For lower frequencies, fatigue life should be reduced, see chapter 3 in CEB Bulletin 188 for guidance.

In the case of compression–tension the criteria for compression as well as the criteria for tension shall be fulfilled.

Compression

For $0 < S_{cd,\min} < 0.8$

$$\log N_1 = (12 + 16 S_{cd,\min} + 8 S_{cd,\min}^2)(1 - S_{cd,\max}) \qquad (6.7\text{–}7a)$$

$$\log N_2 = 0.2 \log N_1 (\log N_1 - 1) \qquad (6.7\text{–}7b)$$

$$\log N_3 = \log N_2 (0.3 - \tfrac{3}{8} S_{Sd,\min}) / \Delta S_{cd} \qquad (6.7\text{–}7c)$$

(*a*) If $\log N_1 \leqslant 6$, then $\log N = \log N_1$
(*b*) If $\log N_1 > 6$ and $\Delta S_{cd} \geqslant 0.3 - \frac{3}{8} S_{cd,\min}$, then $\log N = \log N_2$
(*c*) If $\log N_1 > 6$ and $\Delta S_{cd} < 0.3 - \frac{3}{8} S_{cd,\min}$, then $\log N = \log N_3$

where

$$S_{cd,\max} = \gamma_{Sd} \sigma_{c,\max} \eta_c / f_{cd,fat}$$

$$S_{cd,\min} = \gamma_{Sd} \sigma_{c,\min} \eta_c / f_{cd,fat}$$

$$\Delta S_{cd} = S_{cd,\max} - S_{cd,\min}$$

Tension

$$\log N = 12(1 - S_{td,\max}) \qquad (6.7\text{–}8)$$

where

$$S_{td,\max} = \gamma_{Sd} \sigma_{ct,\max} / f_{ctd,fat}$$

For $S_{cd,\min} \geqslant 0.8$, the S–N relations for $S_{cd,\min} = 0.8$ are valid (see also clause 2.1.7.1).

The value $\log N_3$ is to be calculated only if $\log N_1 > 6$.

For η_c see eq. (6.7–2).
For γ_{Sd} see clause 1.6.4.4.
For the assessment of $\sigma_{c,\max}$, $\sigma_{c,\min}$ and $\sigma_{ct,\max}$ see clauses 1.6.4.2 and 6.7.2 using the fatigue equivalent or frequent value of the variable action Q.
$\sigma_{c,\max}$ and $\sigma_{ct,\max}$ are to be calculated under the upper load effect.
$\sigma_{c,\min}$ is determined as the maximum stress in the compression zone at a distance no more than 300 mm from the surface where $\sigma_{c,\max}$ occurs, but under the lower load effect.

6.7.5. Verification by means of spectrum of load levels

This method takes account of the required lifetime, the load spectrum (which is divided into j blocks) and the characteristic fatigue strength functions.

Fatigue damage D is calculated using the Palmgren-Miner summation

$$D = \sum_{i=1}^{j} \frac{n_{Si}}{N_{Ri}}$$

where

n_{Si} denotes the number of acting stress cycles associated with the stress range for steel and the actual stress levels for concrete
N_{Ri} denotes the number of resisting stress cycles.

The fatigue requirement will be satisfied if $D \leqslant D_{\lim}$.
Using an appropriate counting method (e.g. rainflow method) a value of

The partial coefficients are applied in this procedure as follows.

For steel, values N_{Ri} are calculated from the S–N curves given in Tables 6.7.1 and 6.7.2 using an increased stress range $\gamma_{Sd} \gamma_{s,fat} \Delta\sigma_{Ssi}$.

For concrete, values N_{Ri} are calculated directly from the fatigue strength functions given in subsection 6.7.4.

6.7.6. Shear design

Members without shear reinforcement

The fatigue requirements will be met, if under cyclic loading the required life (number of cycles n) is less than or equal to the numbers of cycles to failure

$$n \leqslant N$$

N should be calculated from the fatigue strength functions given below.

$$\log N = 10(1 - V_{max}/V_{ref}) \tag{6.7–9}$$

where

V_{max} is the maximum shear force under the relevant representative values of permanent loads including prestress and maximum cyclic loading

$V_{ref} = V_{Rd1}$ (see clause 6.4.2.3).

Members with shear reinforcement

The stress in the shear reinforcement should be calculated according to chapter 6 assuming the following inclination of the compression struts under fatigue loading:

$$\tan \theta_{fat} = \sqrt{(\tan \theta)}$$

For assessment of the θ value see subsection 6.3.3.

The resistance of compressive struts can be verified using eq. (6.7–4) or eqs. (6.7–7a), (6.7–7b) and (6.7–7c) reducing the fatigue reference strength given in subsection 6.7.3 by a factor of 0.7. The compression of web concrete subjected to fatigue loading should be calculated using the angle θ (see subsection 6.3.3).

The fatigue reference strength is to be reduced in the same way as the compressive strength of concrete subjected to simultaneously acting compressive and transverse tensile forces. The factor 0.7 applied to the fatigue reference strength corresponds to the reduction of f_{cd1} value (average design stress for cracked compressive zones). $f_{cd2}/f_{cd1} = 0.6/0.85 = 0.7$ (see clause 6.2.2.2).

6.7.7. Increased deflections under fatigue loading

Under cyclic loading progressive deflection can occur in reinforced concrete members in addition to the deflection produced by creep. The cyclic effect can be calculated from

$$a_n = a_1[1.5 - 0.5 \exp(-0.03 n^{0.25})] \tag{6.7–10}$$

where

a_1 is the deflection in the first cycle due to the maximum load including effects of shear strains

n is the number of cycles.

6.8. DEEP BEAMS AND DISCONTINUITY REGIONS

6.8.1. Scope and basic criteria

This section applies to members or parts of members where the assumption of linear strain distribution is not valid.

The verification shall be based on physical models according to the requirements given in section 6.1.

This applies to deep beams, regions of other members with concentrated loads or reactions (statical discontinuities) and regions with geometrical discontinuities, including the connections of different members.

According to section 5.6 the analysis may be carried out by applying

- linear analysis
- analysis by admissible stress fields (truss analogy)
- non-linear analysis.

For truss analogy the model of resistance is an arrangement of compression fields: struts, ties and nodes. The compatibility of deformations should be considered by orientating the models at the force systems given by linear elastic analysis of uncracked members and connections.

The orientation at the linear elastic stress system is more important for the compression struts than for the ties which usually can be arranged parallel to the edges of the member following practical considerations of reinforcement layout. In highly stressed node regions (e.g. near supports or concentrated loads) the main struts and ties of the model should normally meet at angles of about 60° and not less than 45°.

Alternatively, this selection may be based on previous experimental or analytical data available in the literature; however, in such a case, a sensitivity check might be needed for several trial schemes.

It is to be verified that under the action of the design loads the stresses in the compression fields and ties do not exceed the basic strength criteria given in section 6.2, and that the nodes and anchorages comply with section 6.9.

Energy criteria may be used for the selection of the model.

6.8.1.1. Ties

The steel ties have to be dimensioned so that the resistance according to subsection 6.2.4 is not exceeded. The arrangement of the reinforcing bars shall be chosen to avoid disproportionate cracking at the SLS (see section 7.4). The detailing of the bars should follow the rules of chapter 9.

Normally the tensile resistance of concrete should not be relied upon in any major tie (see subsection 6.2.3).

If the arrangement of the reinforcement is made in accordance with a linear elastic stress field a SLS verification can normally be avoided.

Tension in re-entrant corners leads to relatively wide cracks. Additional diagonal bars are recommended there, even if a corresponding tie is not provided in the model or analysis.

6.8.1.2. Concrete compression fields or struts

The concrete dimensions shall be such that the strength criteria given in clause 6.2.2.2 are not exceeded.

In case of linear elastic and non-linear analyses the stress field is directly determined. Further guidance for applying the strength criteria and safety factors is given in chapter 5. Additional to the analysis the anchorages shall be verified.

The stresses in struts of truss models need normally not be verified, if their singular nodes are checked and if reinforcement transverse to the strut axis is provided (see subsection 6.9.1).

The total transverse force between a concentrated node and the bulge of a compression stress field can be assumed not to exceed 25% of the total strut force unless shown otherwise. For struts with concentrated nodes at both ends the transverse reinforcement between the nodes may be designed for 30 to 40% of the strut force, considering stress redistribution after cracking.

Transverse tension due to the deviation of the bulging compression stress trajectories within a stress field or strut requires transverse reinforcement unless the concrete tensile strength is sufficient and reliable enough for carrying the tension (see section 3.3).

6.8.1.3. Nodes and anchorages

It shall be secured that the forces of the struts and ties are balanced in the node region (see subsection 6.1.5) and that the strength criteria for nodes and anchorages, given in section 6.9, are not exceeded.

6.8.2. Examples of application of admissible stress fields

6.8.2.1. Plate elements

Based on a linear-elastic solution of stress pattern, dimensioning may be carried out by means of a truss model, composed of struts and ties as follows.

For continuous deep beams, special consideration should be given to differential settlements of the supports.

(*a*) For the plate-element under consideration, an appropriate number of typical sections is selected approximately perpendicular to the principal stress-trajectories, both compressive and tensile.

(*b*) Across each of these sections an appropriate number of struts and ties are positioned as follows.

- (i) Struts are roughly oriented in alignment with compressive stress-trajectories, and are placed in the respective centres of gravity of these stresses across each selected cross-section.
- (ii) Ties are oriented and placed in a similar way; however, their orientation may be influenced by the practicality of the layout of reinforcement, whereas their position may be dictated by the importance of the expected cracks; thus, ties parallel or perpendicular to the edges of the plate-element are generally preferred.

A further departure from such an orientation may be considered, accounting for redistribution of forces due to the cracked state; such more pragmatic solutions are more economical, provided the SLS is more strictly considered.

(*c*) Axial forces acting along each bar of the truss are subsequently calculated. Verification follows as in clauses 6.8.1.1 and 6.8.1.2.

Single-span deep beams as shown in Fig. 6.8.1 may be designed for chord forces derived from an internal lever arm $z = 0.6\text{–}0.7l$, but not more than the lever arm obtained from a standard analysis of linear members according to section 6.3.

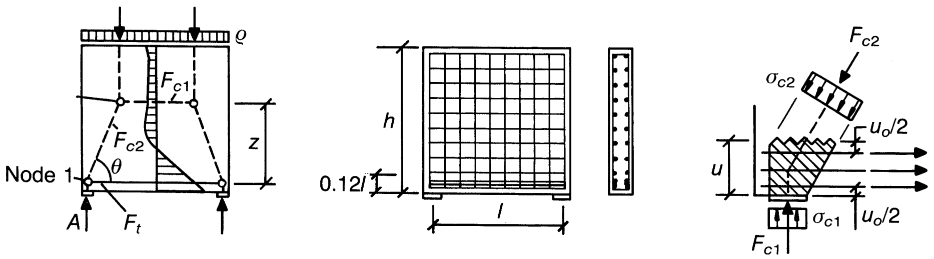


Fig. 6.8.1. Strut-and-tie model, reinforcement and idealized support node for a deep beam

For the deep beam shown in Fig. 6.8.1 the following checks are necessary.

(*a*) Tension chord F_t. Reinforcement according to subsection 6.2.4 for $F_t \approx 0.4A$.

(*b*) Support node 1

(i) Bearing pressure of support according to clause 6.9.2.3 and subsection 6.9.1. Alternatively the pressure σ_{c2} from the strut F_{c2} must be checked, if the height u of the node and the strut angle θ is relatively low ($u < a_1 \cot\theta$).

(ii) Anchorage length according to clause 6.9.2.3 and subsection 6.9.5.

(*c*) Strut F_{c2}. Transverse reinforcement between node 1 and 2 for deviation forces $F_{t2} \approx 0.25F_{c2}$ of the bulging stress field according to clause 6.8.1.2. The nominal mesh reinforcement on both sides may have to be increased in the strut region.

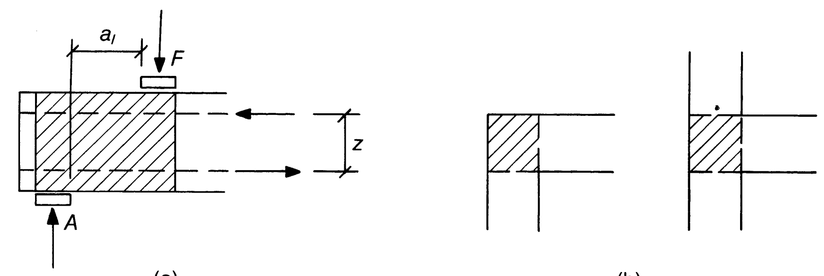
Simplified design procedures are also allowed if based on sufficient evidence and experience.

Special attention shall be paid to dimensioning the anchorage lengths and the nodes at the supports according to section 6.9.

6.8.2.2. Discontinuity regions

6.8.2.2.1 General

This section applies to regions of frames near concentrated forces or regions with geometrical discontinuities (including the connections of different building elements). The design provisions of clause 6.8.2.1 are equally valid in discontinuity regions.



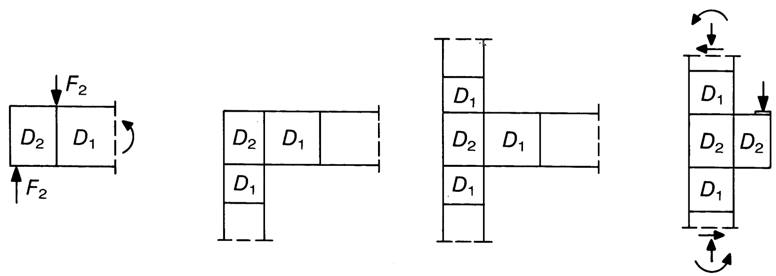
(a) (b)

Fig. 6.8.2. Discontinuity regions: (a) region near concentrated load if $a_l < z \cot \theta$ *(where θ as in clause 6.3.3.1.); (b) geometrical discontinuity*

The technique of 'standard discontinuity regions' may profitably be used to this end.

A broader area is considered, including the 'discontinuity region' (D_2) and nearby 'transition regions' (D_1) where the provisions of subsections 6.3.2 and 6.3.3 are applicable. Input forces coming from D_1-regions (determined as in subsections 6.3.2 and 6.3.3) are acting on the truss model of a D_2-region.

However, simpler techniques may be applied. A direct application of a truss-model may be used, without previous knowledge of an elastic solution of stress-pattern.



(a) (b) (c)

Fig. 6.8.3. Examples for the application of standard discontinuity regions D_1 *and* D_2*: (a) in a beam; (b) in frames; (c) in a corbel*

Standard solutions of truss models in D_2-regions may be derived and subsequently used in several cases; an example is given in Fig. 6.8.4.

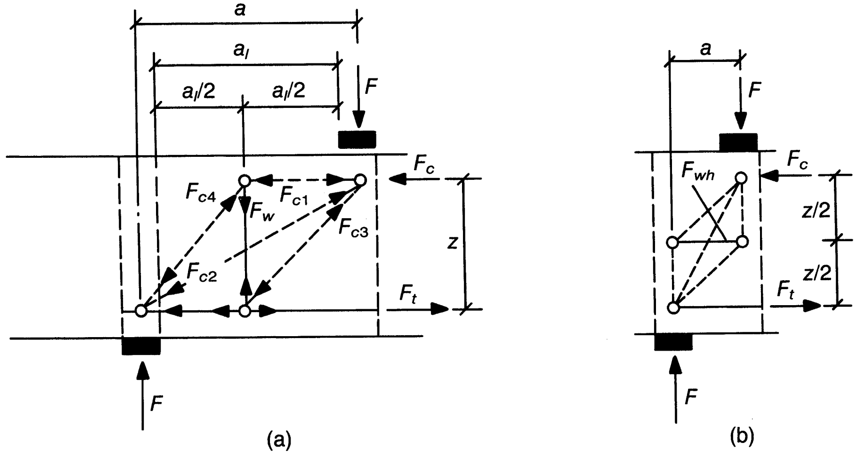


Fig. 6.8.4. Modelling of a standard discontinuity region: (*a*) a $> \frac{1}{2}$z, a $<$ z$\cot\theta$; (*b*) a $< \frac{1}{2}$z

Appropriate stirrups, accommodated close to F_w will secure the transfer of this force. Conservative approximate expressions for F_w as a function of the ratio a/z and of the resultant axial and shear forces may be used, e.g.

$$F_w = \frac{2a/z - 1}{3 - N_{Sd/F}} F \begin{Bmatrix} > 0 \\ < F \end{Bmatrix}$$

$$F_{wh} = \frac{2z/a - 1}{3 + F/F_c} F_c \begin{Bmatrix} > 0 \\ < F_c \end{Bmatrix}$$

where axial tension N_{Sd} (not shown in Fig. 6.8.3) is defined positive.

Appropriate verifications should be carried out within discontinuity regions, regarding

- web reinforcement
- diagonal web forces (if web-width is smaller than the width of the nodes where concentrated forces are acting)
- strut forces acting on nodes
- anchorage of reinforcement in the node regions.

Reference is made to subsection 6.9.2.

High beam/narrow column ($h_2/h_1 > 1.5$)

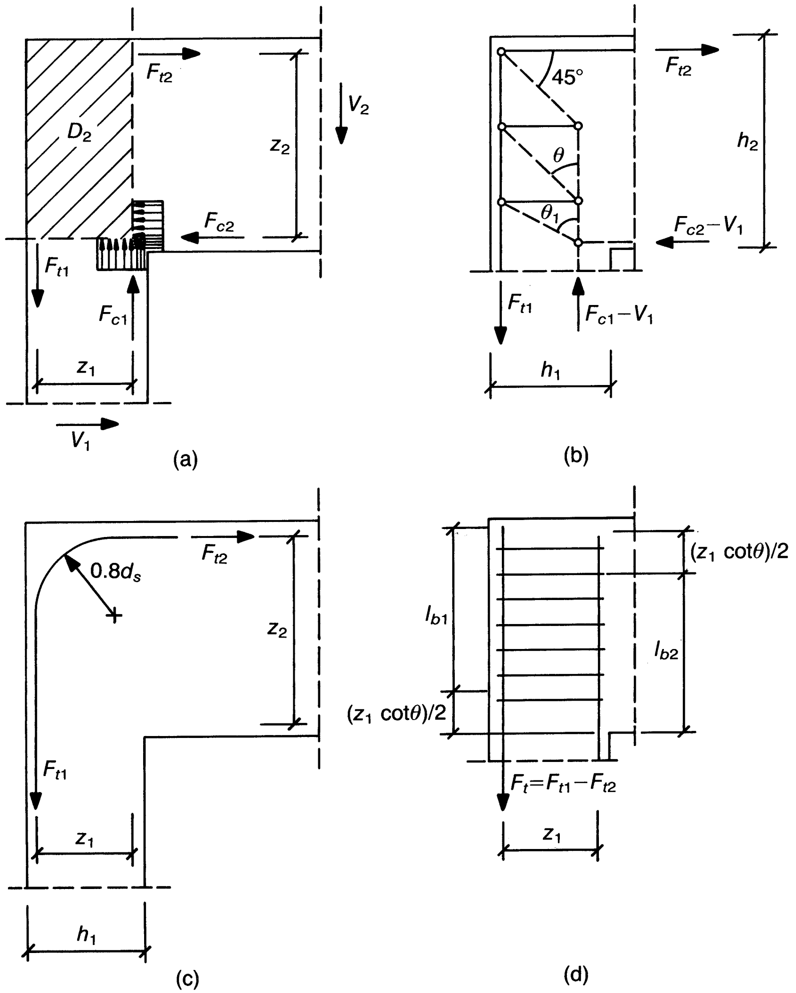


Fig. 6.8.5. Frame corner with closing moments: (a) input forces; (b) truss model; (c) bent around bars; (d) anchorage lengths of column reinforcement, l_{b1} *(tensile) and* l_{b2} *(compressive)*

6.8.2.2.2. Frame corners with closing moments

It should be secured that all input forces acting on the beam-column joint area will be safely transferred through the body of the joint.

Assuming a strut angle $\theta = 45°$, a safe value of the total horizontal force F_w of the stirrup reinforcement is given by $F_w = F_{t1} - F_{t2}$. Appropriate anchorage lengths should be secured: l_{b1} for a tensile force $F_{11} - F_{12}$ (outer reinforcement), l_{b1} for the compressive force of inner column reinforcement.

Approximately equal depths of column and beam
If

$$\frac{2}{3} < \frac{h_2}{h_1} < \frac{3}{2}$$

no check of stirrup reinforcement or anchorage lengths within the beam-column joint is needed, provided that all the longitudinal reinforcement of the beam and column is bent around the corner.

The standard solutions given in Fig. 6.8.3 may be applied for the cantilevering part of the structure and its support region.

The internal lever arm z of the models, the main tensile chord force F_t and the compression node 1 (see clause 6.9.2.2) can be checked by application of standard methods for axial action effects (subsection 6.3.2). The critical design section for these verifications (corresponding to the boundary line of the D_2-region shown in Fig. 6.8.3) is defined by x_1 in Figs 6.8.6 and 6.8.7.

Node 2 in Fig. 6.8.6 is of the type shown in Fig. 6.9.3(b) and can be checked accordingly if the diagonal compression struts F_{c2} and F_{c3} are replaced by their resultant. This resultant force is inclined at the angle θ_r derived from

$$\cot \theta_r = \frac{a}{z}\left(1 - \frac{F_w}{2F_v}\right)$$

For the anchorage of main reinforcement in node 2, it is recommended to use relatively small diameter bars in form of horizontal U-loops in several layers (see clause 6.9.2.3). Instead of loops, the bars may end with anchor plates or with a welded transverse bar of equal diameter near the outer surface. For bars bent in the vertical plane the anchorage length begins below the inner edge of the loading plate.

6.8.2.2.3. Corbels

Corbels shall be designed using truss models. If $a > z$ the corbel may be designed as a linear member (section 6.3).

Normally, in addition to the main chord reinforcement, closed horizontal stirrups shall be provided if $a < z/2$ and closed vertical stirrups if $a > z/2$ or a combination of both in all cases $a < z$. Instead, inclined stirrups can be used where suitable (e.g. Fig. 6.8.7).

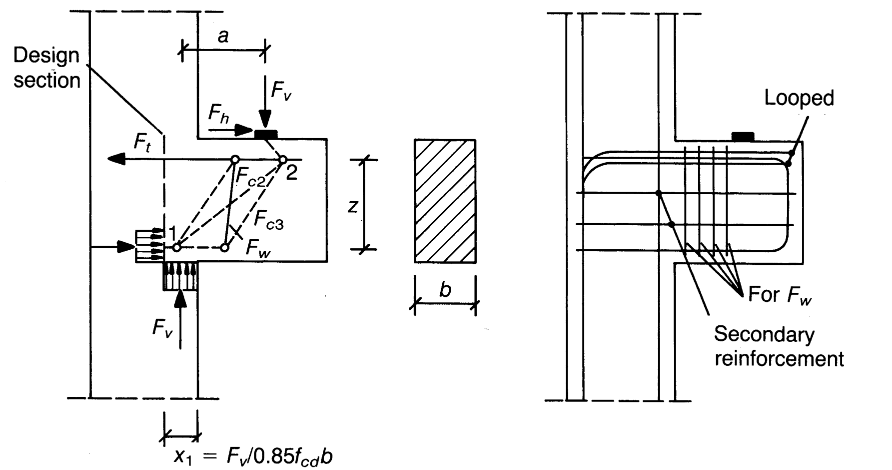


Fig. 6.8.6. Truss model and reinforcement for a corbel (compare Fig. 6.8.3)

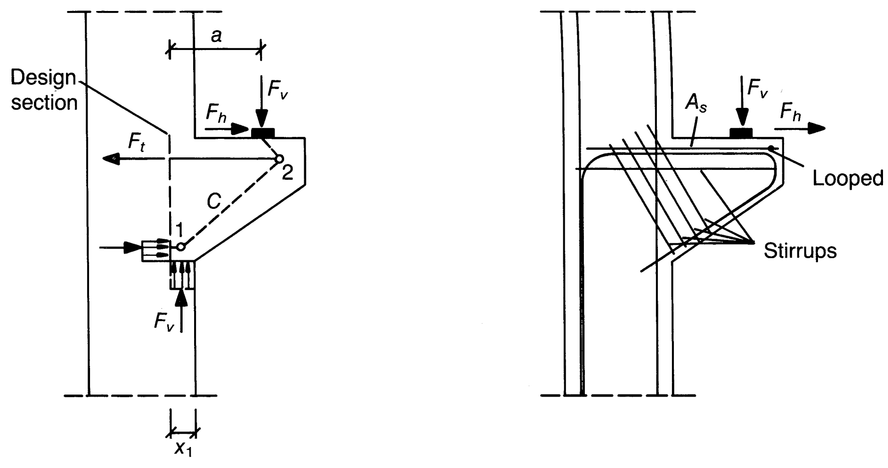


Fig. 6.8.7. Simple strut-and-tie model and reinforcement for a corbel

6.9. VERIFICATION OF NODES AND ANCHORAGES

6.9.1. General

A node is defined as a volume of concrete contained within the intersections between compression fields of struts, in combination with anchorage forces and/or external compressive forces (imposed loads or support reactions).

The nodes should be dimensioned so that all forces are anchored and balanced safely.

Examples of typical nodes are given in subsection 6.9.2.

Normally, the geometry of acting forces and the directions of struts allow for the determination of the dimensions of a node.

In areas of large supports, the distribution of strains should possibly be known or approximately estimated in order to determine the dimensions of nodes (especially in areas of non-rotative supports).

The geometry of the node region and the arrangement of reinforcement in it should be consistent with the model on which the design of the structure is based and with the applied forces. Thereby the equilibrium conditions should be fulfilled.

Nodes should be verified accordingly

- verification of the stresses from the compressive struts in the node according to subsection 6.9.2
- verification of the anchorages of ties according to subsection 6.9.3 and following.

The compressive stresses of the adjoining struts of the node should be within the design limits specified in section 6.2.

Normally the compressive stresses of nodes need to be checked only where concentrated forces are applied to the surface of the structural element, e.g. below bearing plates, anchor plates and over supports. A

verification of node pressures within the structure may become necessary at geometrical discontinuities (holes, corners). Often such node pressures near discontinuities can be checked well enough by the application of standard methods for linear members (subsection 6.3.2) on a section through the whole member; in such a case the node pressures replace the compressive stresses due to the respective bending moment. The design strength for the compression zone should in these cases comply with subsection 6.9.2.

If the length of the bar in the node region is less than the anchorage length required in subsection 6.9.5, the anchored bar may be extended beyond the node region and thus introduce part of its force into the node by compression from behind (see Fig. 6.9.3(b)). In this case tension bars and compression stresses behind the node work like a bowden cable with no perceptible resultant force in the model.

These transverse tensile forces are orthogonal to the plane in which Figs 6.9.2–6.9.5 are drawn.

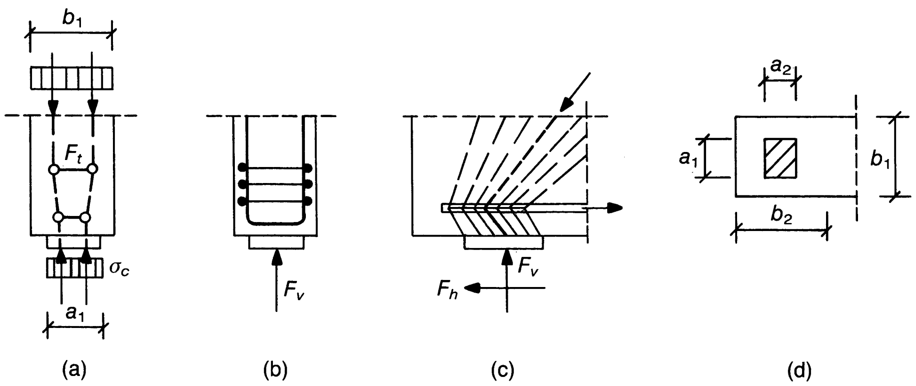


Fig. 6.9.1. Support area: (a) structural model; (b) reinforcement layout; (c) horizontal force at support; (d) homologous areas

For concentrated loads as shown in Fig. 6.9.1 the design bearing pressure can be increased by a factor

$$\beta = \min \begin{cases} b_1/a_1 \\ b_2/a_2 \end{cases} > 4$$

The anchorage of bars shall comply with subsection 6.9.5. The anchorage length will be assumed to begin at the section where the transverse compressive stress trajectories of a strut meet the anchored bar and are deviated. The anchored bar shall extend at least over the whole length of the compression field which is deviated by it.

Minimum radii of bent bars according to clause 9.1.1.2 shall be observed in order to limit the concrete stresses in the supported strut.

Transverse tensile forces from bond action and minor non-uniformities of applied strut stresses should normally be covered by structural reinforcement (e.g. stirrups) which is arranged near the surfaces.

6.9.2. Standard cases of nodes

6.9.2.1. Bearing stresses and other node stresses

Average design stresses in any surface or section through a singular node shall normally not exceed the following values of concrete strength

f_{cd1} for nodes where only compression struts meet
f_{cd2} for nodes where main tensile bars are anchored.

f_{cd1} and f_{cd2} are given in clause 6.2.2.2 for essentially uniaxial compression but can be applied also to nodes with multiaxial states of stress.

f_{cd1} may also be applied in other nodes if the angle between ties and major struts is not less than 55° and if the reinforcement layout in the node region is designed with special care (e.g. arranged in several layers, with transverse ties).

For nodes with secured triaxial compression, e.g. due to local compression or due to lateral confinement by reinforcement, the increased strength values given in section 3.3 for local compression or in clause 2.1.3.4 for multiaxial states of stress may be applied to individual node surfaces (e.g. at bearings), provided that all tensile forces in the node regions and adjacent struts are carried by reinforcement.

where

a_1 and a_2 are the dimensions of the loaded area (Fig. 6.9.1)
b_1 and b_2 (homologous to the loaded area) are determined from limitations to the dispersion of the stresses (Fig. 6.9.1).

Transverse tension in the case (Fig. 6.9.1(a)) may be estimated from the formula

$$F_t = \frac{1}{4} \frac{b_1 - a_1}{b_1} F_v \qquad (6.9\text{–}1)$$

If an additional horizontal force F_h is acting at a support (Fig. 6.9.1(c)), the acting stress may be estimated using

$$\sigma_c = \frac{F_v}{a_1 a_2} \sqrt{[1 + (F_h/F_v)^2]} \qquad (6.9\text{–}2)$$

Ducts crossing the node region shall be considered like ducts in compression struts (see clause 6.2.2.4).

6.9.2.2. Nodes with only compressive forces

Such nodes occur, for example, under concentrated loads or over inner supports of continuous beams or at supports where prestressing tendons are anchored or in compressed re-entrant corners.

The node region may be assumed to be limited by a polygon not necessarily at right angles to the strut direction. The stresses along the individual node surface can normally be assumed evenly distributed.

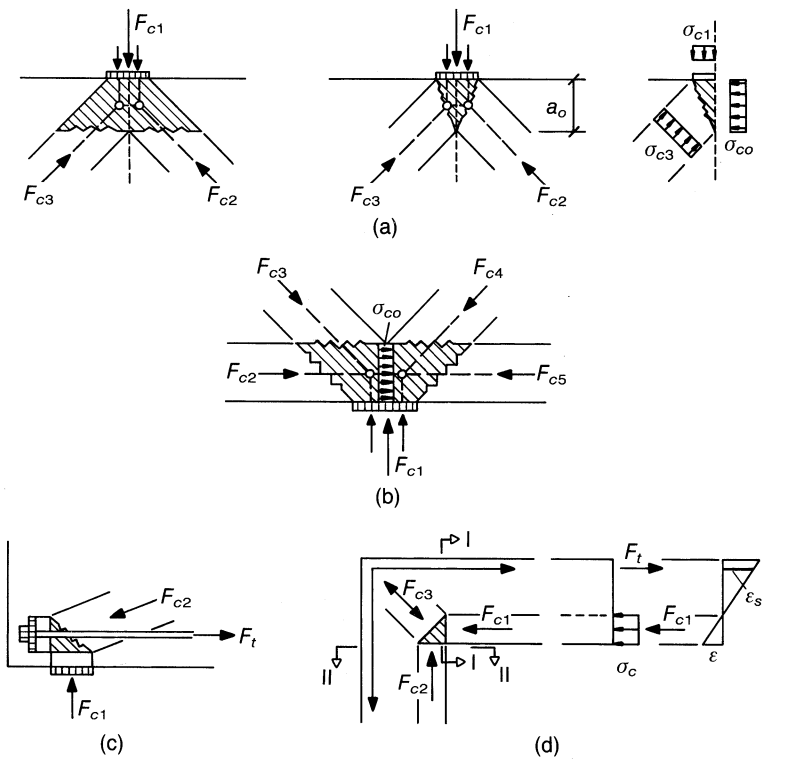


Fig. 6.9.2. Nodes with only compression forces: (a) node under concentrated load; two alternative layouts of the node region with identical pressures; (b) node over inner support of continuous beam; (c) node over end support of prestressed beam; (d) node in compressed re-entrant corner with simplified check of node pressure in section I–I assuming linear strain distribution

In the cases of Fig. 6.9.2(a) and (b) it is normally sufficient to check only the bearing pressure with respect to f_{cd1}. However, if the height a_o of the nodes is restricted by a crack or by the height of the on-coming struts F_{c2} and F_{c5} as is the case for the compression chords of beams, the pressure σ_{co} in the section orthogonal to the bearing should also be verified, e.g. by standard methods for beams (subsection 6.3.2). Accordingly in the cases of Fig. 6.9.2(c) and (d) the pressures in both orthogonal faces of the node should be checked.

6.9.2.3. Nodes with anchorage of parallel bars only (compression-tension node)

Such nodes occur where in the strut-and-tie model one tie meets two or more compression struts, e.g. at end-supports and below concentrated loads which are applied to corbels or near the corner of deep beams. They further occur frequently in regions where one-dimensional members are connected to other members and apply concentrated chord forces on them, e.g. in frame corners with opening moments and in beam-column-connections.

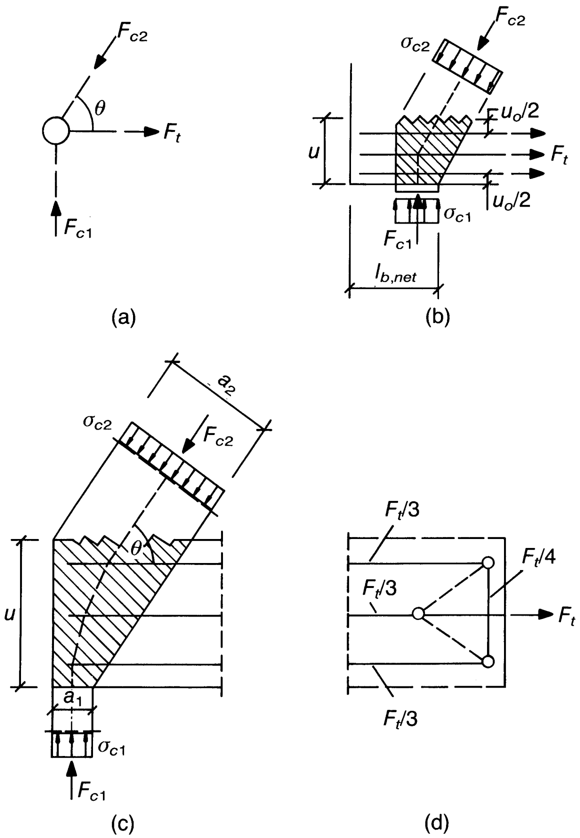


Fig. 6.9.3. Node with anchorage of parallel bars only: (a) scheme; (b) node with reinforcement extended beyond the bearing; (c) node with bars ending within the node region; (d) distribution of a concentrated tie F_t *over several layers of reinforcement (stirrups, loops)*

Detailing of the reinforcement is essential in such nodes (Fig. 6.9.3). The reinforcement shall preferably be distributed over the height u in several layers and anchored by loops or hooks which are bent in the plane orthogonal to the compression F_{c1}.

Normally the tie force F_t cannot be anchored within the short length of the node if the reinforcement capacity is fully used and if the bars cannot be extended beyond the node, as e.g. in member connections. In such cases

it is without further check of anchorage lengths sufficient to transfer 2/3 of the tie force to parallel bars (stirrups or loops) which are arranged in several layers (Fig. 6.9.3(c) and (d)) in order to extend the node region over a greater height u and to reduce the individual anchor forces of the bars. The transverse tie force due to the lap requires additional transverse reinforcement for approximately $F_t/4$, unless this force is compensated by compression.

The transverse pressure reduces the design anchorage length according to subsection 6.9.5.

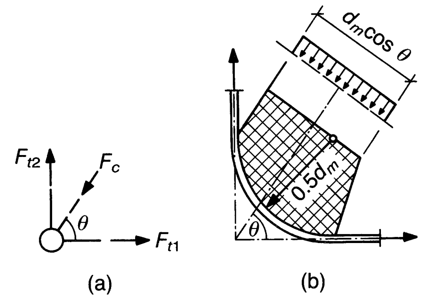


Fig. 6.9.4. Node with bent bars

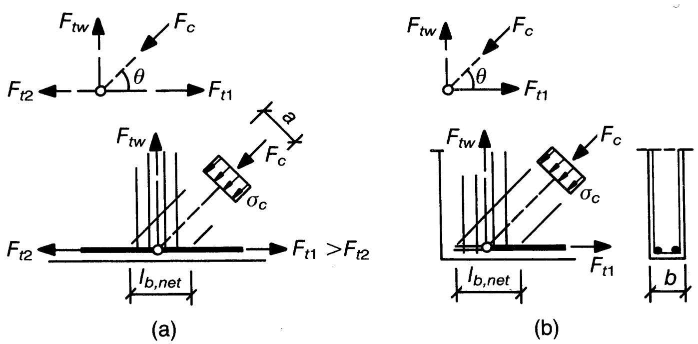


Fig. 6.9.5. Nodes with ties in orthogonal directions: (a) node in a chord at the edge of a member; (b) node in the corner of a member

6.9.2.4. Nodes with bent bars

Such nodes occur where a strut force is balanced mainly by the deviation forces of bent bars and also by bond forces if the node is unsymmetric with respect to the strut.

The radius of bend shall comply with clause 9.1.1.2.

Bars bent around a rectangular corner tend to anchor the diagonal strut partly ahead of the bend by bond similar to the node described in clause 6.9.2.5. Therefore transverse ties (stirrups, loops) should be arranged immediately before and after the bend.

6.9.2.5. Nodes with ties in orthogonal directions

Such nodes typically occur distributed over considerable lengths of bars in the edges and corners of members, if bond forces are applied to them, e.g. in the tension chords of beams or deep beams and in the chords of standard discontinuity regions (clause 6.8.2.2).

The diagonal compression stresses in the node need normally not be verified if the anchorage length or bond stresses comply with subsections 6.9.2–6.9.5. However, if a multi-layered reinforcement chord is anchored, the diagonal concrete stresses may become critical. They have to comply with the appropriate design strengths.

The reinforcement F_{tw} orthogonal to the edge of the member can be anchored only by hooks or bends (stirrups, loops) which enclose the longitudinal bars (chord reinforcement). Therefore small diameter bars at small spacings should be chosen for F_{tw}. The detailing rules for shear reinforcement in beams (clause 9.2.2.2) and the anchorage of stirrups and shear assemblies should be observed accordingly.

6.9.3. Design bond stress for reinforcing bars

The design value of the bond stress f_{bd} is

$$f_{bd} = \eta_1 \eta_2 \eta_3 f_{ctd} \tag{6.9–4}$$

where

f_{ctd} is the design value of concrete tensile strength $(= f_{ctk,\min}/1.50)$
η_1 considers the type of reinforcement: $\eta_1 = 1.0$ for plain bars, $\eta_1 = 1.4$ for indented bars and $\eta_1 = 2.25$ for ribbed bars

η_2 considers the position of the bar during concreting: $\eta_2 = 1.0$ when good bond conditions are obtained, as for

- all bars with an inclination of 45°–90° to the horizontal during concreting and
- all bars with an inclination less than 45° to the horizontal, which are up to 250 mm from the bottom or at least 300 mm from the top of the concrete layer during concreting;

$\eta_2 = 0.7$ for all other cases and for bars in structural parts built with slip forms

η_3 considers the bar diameter

$$\eta_3 = 1.0 \quad \text{for} \quad \phi \leqslant 32\,\text{mm}$$

$$\eta_3 = \frac{132 - \phi}{100} \quad \text{for} \quad \phi > 32\,\text{mm}$$

with ϕ in mm.

6.9.4. Basic anchorage length

The basic length necessary for the transfer of the yield force of a bar or wire of diameter ϕ is

$$l_b = \frac{\phi}{4}\frac{f_{yd}}{f_{bd}} \qquad (6.9\text{–}5)$$

f_{bd} having the values defined in subsection 6.9.3.

For a welded mesh made up of plain or indented wires, l_b in eq. (6.9–5) can be calculated with the values for f_{bd} obtained with η_1 for ribbed bars in subsection 6.9.3, provided there is a sufficient number of welded cross bars in the anchorage zone (see subsection 6.9.5).

Each welded joint should be capable of withstanding the shearing force given in clause 2.2.5.1b.

6.9.5. Design anchorage length

The design anchorage length $l_{b,net}$ can be calculated from eq. (6.9–6)

$$l_{b,net} = \alpha_1 \alpha_2 \alpha_3 \alpha_4 \alpha_5 l_b A_{s,cal}/A_{s,ef} \geqslant l_{b,\min} \qquad (6.9\text{–}6)$$

where

$A_{s,cal}$ is the calculated area of reinforcement required by the design
$A_{s,ef}$ is the area of reinforcement provided.
α_1, α_2, α_3, α_4, α_5 are coefficients given by Table 6.9.1 and defined as

- α_1 coefficient taking into account the form of the bar (straight, bent, loop)
- α_2 coefficient taking into account the influence of one or more welded transverse bars ($\phi_l > 0.6\phi$) along the design anchorage length $l_{b,net}$ (see Fig. 6.9.6)
- α_3 coefficient taking into account the effect of confinement by the concrete cover
- α_4 coefficient taking into account the effect of confinement by transverse reinforcement
- α_5 coefficient taking into consideration the effect of the pressure transverse to the plane of splitting along the design anchorage length

l_b is taken from eq. (6.9–5)

The effect of other types of anchoring devices may be taken into account in the same way. The appropriate α_2-value should be determined by tests.

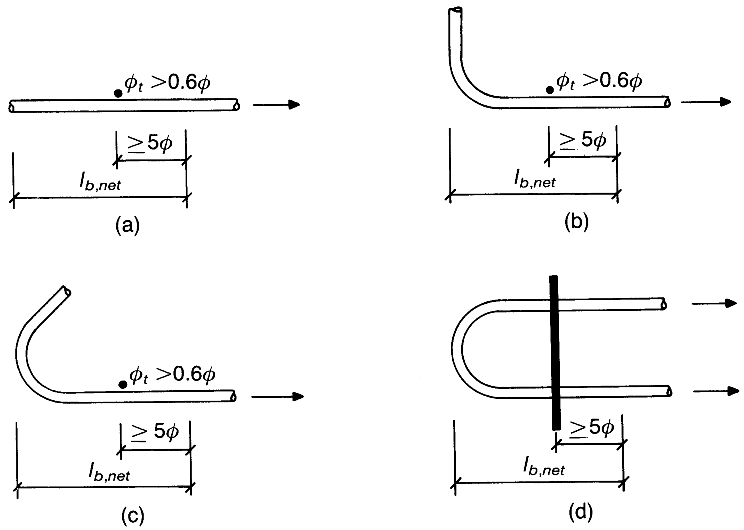


Fig. 6.9.6. Requirement on position of the welded transverse bar along $l_{b,net}$

The limitations of $l_{b,\min}$ are given

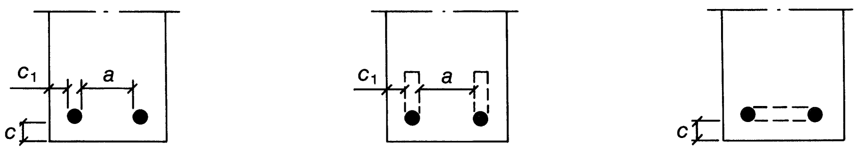
- to ensure minimum active anchorage length
- to take into account tolerances.

$l_{b,\min}$ denotes the minimum anchorage length:

- for bars in tension: $l_{b,\min} > \max\{0.3l_b; 10\phi; 100\,\text{mm}\}$
- for bars in compression: $l_{b,\min} > \max\{0.6l_b; 10\phi; 100\,\text{mm}\}$.

The product $(\alpha_3\alpha_4\alpha_5)$ is limited:

— for high-bond bars: $\alpha_3\alpha_4\alpha_5 > 0.7$,
— for plain or indented bars or wires: $\alpha_3\alpha_4\alpha_5 = 1$.



c_d = min (a/2, c_1, c) (a)

c_d = min (a/2, c_1) (b)

$c_d = c$ (c)

Fig. 6.9.7. Notation for Table 6.9.1: (a) straight bars; (b) hooks, bends; (c) loops, horizontally placed hooks, bends

Minimum concrete cover for the bond model to be valid is one bar diameter.

The transverse reinforcement should be evenly distributed along

- $l_{b,net}$ in the case of tensile anchorages
- $l_{b,net} + 4\phi$ in the case of compression anchorages; at least one unit of the transverse reinforcement should be placed along this 4ϕ region outside the anchored bar.

At least one unit of the transverse reinforcement should be placed in the region of a hook, a bend or a loop.

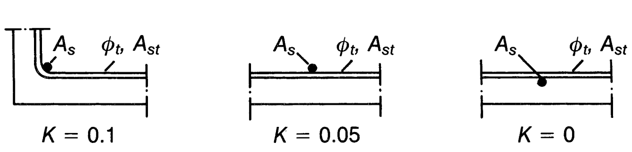
Table 6.9.1. α_1, α_2, α_3, α_4, α_5 coefficients

Influencing factor	Type of anchorage	Reinforcement bar		
		In tension		In compression
Form of bars	—	$\alpha_1 = 1.0$		$\alpha_1 = 1.0$
		$\alpha_1 = 0.7$[(a)]		$\alpha_1 = 1.0$
Welded transverse bars		$\alpha_2 = 0.7$		$\alpha_2 = 0.7$
Confinement by concrete	—	$\alpha_3 = 1 - 0.15\dfrac{c_d - \phi}{\phi}$	$\begin{cases} \geqslant 0.7 \\ \leqslant 1.0 \end{cases}$	$\alpha_3 = 1.0$
		$\alpha_3 = 1 - 0.15\dfrac{c_d - 3\phi}{\phi}$	$\begin{cases} \geqslant 0.7 \\ \leqslant 1.0 \end{cases}$	$\alpha_3 = 1.0$
Confinement by not welded transverse reinforcement	—	$\alpha_4 = 1 - K\lambda$[(b)]	$\begin{cases} \geqslant 0.7 \\ \leqslant 1.0 \end{cases}$	$\alpha_4 = 1.0$
Confinement by transverse pressure	—	$\alpha_5 = 1 - 0.04p$	$\begin{cases} \geqslant 0.7 \\ \leqslant 1.0 \end{cases}$	

[(a)] If $c_d > 3\phi$, otherwise $\alpha_1 = 1.0$.
[(b)] $\lambda = (\Sigma A_{st} - \Sigma A_{st,\min})/A_s$.

Notation:

ΣA_{st} is the cross-sectional area of the transverse reinforcement along the design anchorage length $l_{b,net}$
$\Sigma A_{st,\min}$ is the cross-sectional area of the minimum transverse reinforcement $= 0.25A_s$ for beams and 0 for slabs
A_s is the area of a single anchored bar with maximum bar diameter
K values are



p is the transverse pressure (MPa) at ultimate limit state along $l_{b,net}$, perpendicular to splitting plane.

Anchorage of stirrups is given in clause 9.1.1.4.

For the shear strength of welds, see clause 2.2.5.1b.

For a welded mesh made of plain or indented wires, the design anchorage length $l_{b,net}$ can be calculated as for mesh made of high-bond wires, provided the number of welded cross wires over the design anchorage length is

$$n = 4A_{s,cal}/A_{s,ef}$$

6.9.6. Design lap length of bars in tension

The design lap length is

$$l_0 = \alpha_1\alpha_3\alpha_4\alpha_5\alpha_6 l_b A_{s,cal}/A_{s,ef} \geqslant l_{0,\min} \quad (6.9\text{–}7)$$

where

l_b is calculated from eq. (6.9–5)
$l_{0,\min} > \max\{0.3\alpha_6 l_b;\ 15\phi;\ 200\,\text{mm}\}$
α_1, α_3, α_4 and α_5 can be taken from Table 6.9.1,

however, for the calculation of α_4, $\Sigma A_{st,\min}$ should be taken as $1.0A_s$, with A_s = area of one spliced bar

α_6 is a coefficient given in Table 6.9.2 as a function of the percentage of the reinforcement lapped within $1.3l_0$ from the centre of the lap length considered.

For transverse distribution reinforcement α_6 can be taken equal to 1.0.

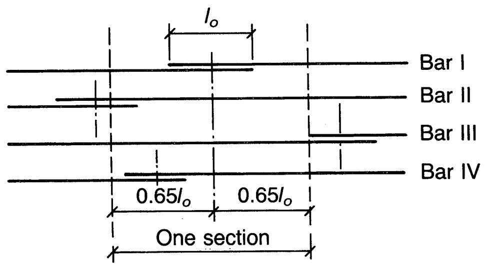


Fig. 6.9.8. Percentage of lapped bars in one section

Table 6.9.2. Values of the coefficient α_6

Percentage of lapped bars relative to the total cross-section of steel	$\leqslant 20\%$	25%	33%	50%	$> 50\%$
α_6	1.2	1.4	1.6	1.8	2.0

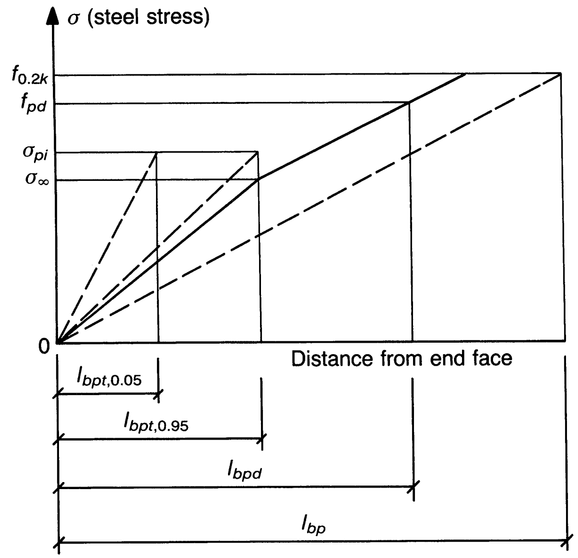


Fig. 6.9.9. Course of steel stresses along the anchorage zone of a pretensioned member

6.9.11.2. Design bond strength

The design value of the bond strength for prestressing tendons is

$$f_{bpd} = \eta_{p1}\eta_{p2}f_{ctd} \tag{6.9–12}$$

where

$f_{ctd} = f_{ctk}(t)/1.50$ is the lower design concrete tensile strength; for the transmission length the strength at the time of release, for the anchorage length the strength at 28 days

η_{p1} takes into account the type of prestressing tendon: $\eta_{p1} = 1.4$ for indented and crimped wires, and $\eta_{p1} = 1.2$ for 7-wire strands

η_{p2} takes into account the position of the tendon: $\eta_{p2} = 1.0$ for all tendons with an inclination of 45°–90° with respect to the horizontal during concreting, $\eta_{p2} = 1.0$ for all horizontal tendons which are up to 250 mm from the bottom or at least 300 mm below the top of the concrete section during concreting, and $\eta_{p2} = 0.7$ for all other cases.

6.9.11.3. Basic anchorage length

The basic anchorage length of an individual pretensioned tendon is

$$l_{bp} = \frac{A_{sp}}{\phi\pi}\frac{f_{ptd}}{f_{bpd}} \qquad (6.9\text{–}13)$$

$f_{ptd} = f_{ptk}/1.15$, where f_{ptk} as defined in clause 2.3.4.3.

The basic anchorage length defines the length that is required to develop the full strength in an untensioned tendon.

The factor $A_{sp}/\phi\pi$ depends on the type of tendon.

For tendons with a circular cross-section

$$\frac{A_{sp}}{\phi\pi} = \frac{\phi}{4}$$

For 7-wire strands

$$\frac{A_{sp}}{\phi\pi} = \frac{7}{36}\phi$$

6.9.11.4. Transmission length

The transmission length of a pretensioned tendon is

$$l_{bpt} = \alpha_8\alpha_9\alpha_{10}l_{bp}\frac{\sigma_{pi}}{f_{pd}} \qquad (6.9\text{–}14)$$

where

α_8 considers the way of release: $\alpha_8 = 1.0$ for gradual release and $\alpha_8 = 1.25$ for sudden release;

α_9 considers the action effect to be verified: $\alpha_9 = 1.0$ for calculation of anchorage length when moment and shear capacity is considered, and $\alpha_9 = 0.5$ for verification of transverse stresses in anchorage zone

α_{10} considers the influence of bond situation: $\alpha_{10} = 0.5$ for strands and $\alpha_{10} = 0.7$ for indented or crimped wires

σ_{pi} is the steel stress just after release.

The use of narrow spaced stirrups or helices around the tendons and transverse prestressing may result in a shorter transmission length. This is not considered due to lack of experimental data.

Tendon release that is obtained by sawing through the concrete and the steel should be considered as gradual release.

The transmission length can be estimated from the draw-in value (δ_e) of the tendons at the end face of the concrete member. Assuming a linear steel stress along the transmission length, this draw-in shall be

$$\delta_e < 0.5\frac{\sigma_{pi}}{t_p}l_{bpt}$$

with $\alpha_9 = 1.0$ in the expression for l_{bpt}.

When the concrete member is sawn from a longer production unit, the draw-in cannot be estimated properly.

See commentary to clause 6.9.11.1 for different bond situations. The basic anchorage length is related to 'pull-out'. The transmission length is connected to 'push-in'. The ratio between the two is given by α_{10}.

6.9.11.5. Design anchorage length

The design anchorage length of a pretensioned prestressing tendon is

$$l_{bpd} = l_{bpt} + l_{bp}\frac{\sigma_{pd} - \sigma_{pcs}}{f_{pd}} \qquad (6.9\text{–}15)$$

If necessary, the required anchorage capacity may be obtained by additional end anchorages or non-prestressed reinforcement.

where

σ_{pd} tendon stress under design load ($\sigma_{pd} \leqslant f_{pd}$)
σ_{pcs} tendon stress due to prestress including all losses.

6.9.11.6. Development length

The development length is the distance from the end face to the concrete cross-section beyond which the distribution of the longitudinal stresses is considered linear.

For a rectangular cross-section and straight tendons situated near the bottom edge of the concrete section the development length is

$$l_p = \sqrt{[h^2 + (0.6 l_{bpt})^2]} > l_{bpt} \qquad (6.9\text{–}16)$$

where h is the total depth of the concrete section.

For non-rectangular sections the development length can be found in a similar way as assumed for post-tensioning (clause 9.1.6.1).

6.9.12. Transverse stresses in the anchorage zone of prestressed tendons

6.9.12.1. General

The anchorage zone of prestressed tendons is a discontinuity region that should be treated according to section 6.8. Should the use of the strut-and-tie model be too problematic because of the complexity of the stress field, the verification may be performed on the basis of the stresses in a linear, uncracked member. For design purposes, the tensile stresses, due to the development and distribution of the prestressing force, are subdivided into three groups (Fig. 6.9.10).

If the strut-and-tie model is not applicable due to lack of transverse reinforcement, the verification may be performed on the basis of stress and strain analysis.

6.9.12.2. Bursting

For the calculation of the bursting force the symmetric prism analogy may be used (Fig. 6.9.11). The height and the width of the prism follow from the possible enlargement of the anchor plates (post-tensioning) or the tendon pattern (pretensioning). For multiple tendons the most unfavourable situ-

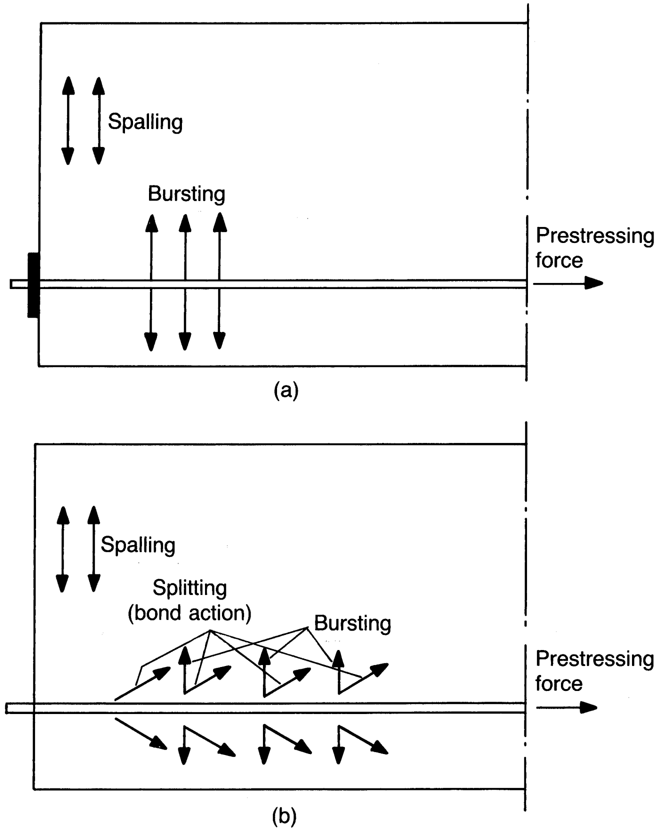


Fig. 6.9.10. Transverse tensile stresses in the anchorage zone: (a) of a post-tensioned member; (b) of a pretensioned member

It should be realized that splitting—in case of pretensioned tendons—and

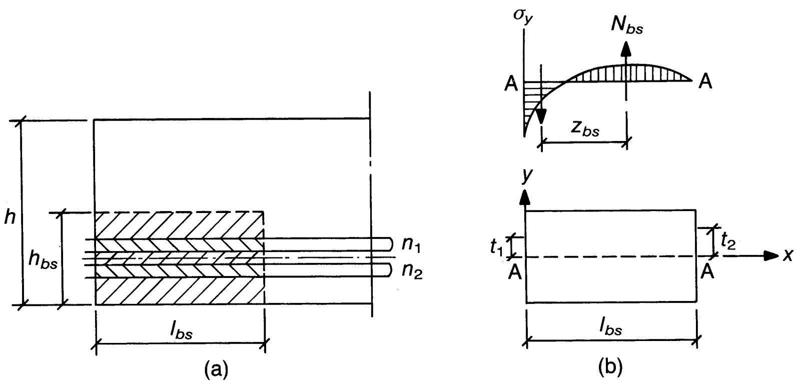


Fig. 6.9.11. For calculation of the bursting force: (a) dimensions of the symmetrical prism; (b) moment equilibrium along section A–A

bursting action shall be determined both in the vertical and in the horizontal direction.

The length of the prism is for end anchored tendons

$$l_{bs} = h_{bs}$$

and for tendons anchored by bond

$$l_{bs} = \sqrt{[h_{bs}^2 + (0.6 l_{bpt})^2]} < l_{bpt}$$

The internal lever arm for the bursting force is

$$z_{bs} = 0.5 l_{bs}$$

The bursting force follows from the moment equilibrium along section A-A (Fig. 6.9.11)

$$N_{bs} = \frac{\frac{1}{2}(n_1 + n_2)t_2 - n_1 t_1}{z_{bs}} \gamma_1 F_{sd} \qquad (6.9\text{–}17)$$

where

- t_1 is the distance between the centroid of tendons above section A-A to the centroid of the prism
- t_2 is the distance between the centroid of the concrete stress block above section A-A to the centroid of the prism
- n_1, n_2 are the numbers of tendons above and below section A-A, respectively
- F_{sd} is the design force per tendon
- $\gamma_1 = 1.1$ is the supplementary partial safety factor against overstressing by overpumping.

The maximum bursting stress follows from

$$\sigma_{bs} = 2N_{bs}/b_{bs}l_{bs} \qquad (6.9\text{–}18)$$

where b_{bs} is the width of the prism.

The model is based on uncracked concrete but it is sufficiently accurate to be used also for cracked concrete with tensile forces resisted by reinforcement.

For $\sigma_{bs} > f_{ctd}$ the bursting force shall be resisted by confining or net reinforcement distributed within $l_{bs}/3$ to l_{bs} from the end face, with

$$A_{sbs} = N_{bs}/f_{sy} \qquad (6.9\text{–}19)$$

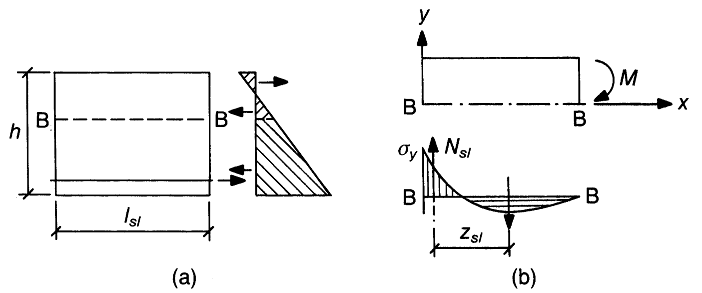


Fig. 6.9.12. For calculation of the spalling force: (a) definition of the equivalent prism; (b) moment equilibrium along section B–B

The equivalent prism approach overestimates the spalling stress. For shallow members (i.e. hollow core slabs) a more accurate value may be obtained from Fig. 6.9.13.

6.9.12.3. Spalling

The spalling force may be calculated with the equivalent prism analogy (Fig. 6.9.12). The length of the prism is defined as, for end anchored tendons

$$l_{sl} = h$$

and for tendons anchored by bond

$$l_{sl} = \sqrt{[h^2 + (0.6 l_{bpt})^2]} < l_{bpt}$$

The internal lever arm for the spalling force is

$$z_{sl} = 0.5 l_{sl}$$

Section **B-B** shall be chosen so that along this section no shear force results. The spalling force results from the moment equilibrium along section **B-B**

$$N_{sl} = M / z_{sl} \qquad (6.9\text{–}20)$$

with the moment M given by the concrete stresses above section **B-B**.

The maximum spalling stress follows from

$$\sigma_{sl} = 8 N_{sl} / b_{sl} l_{sl} \qquad (6.9\text{–}21)$$

with b_{sl} width of the cross-section at section **B-B**.
For $\sigma_{sl} \geqslant f_{ct,fl} / \gamma_c$, where

$\gamma_c = 1.5$
$f_{ct,fl}$ is the flexural tensile strength (given in clause 2.1.3.3.1)

the spalling force shall be resisted by the reinforcement

$$A_{s,sl} = N_{sl} / f_{sy}$$

The spalling force resisting reinforcement shall be put parallel to the end face in its close vicinity.

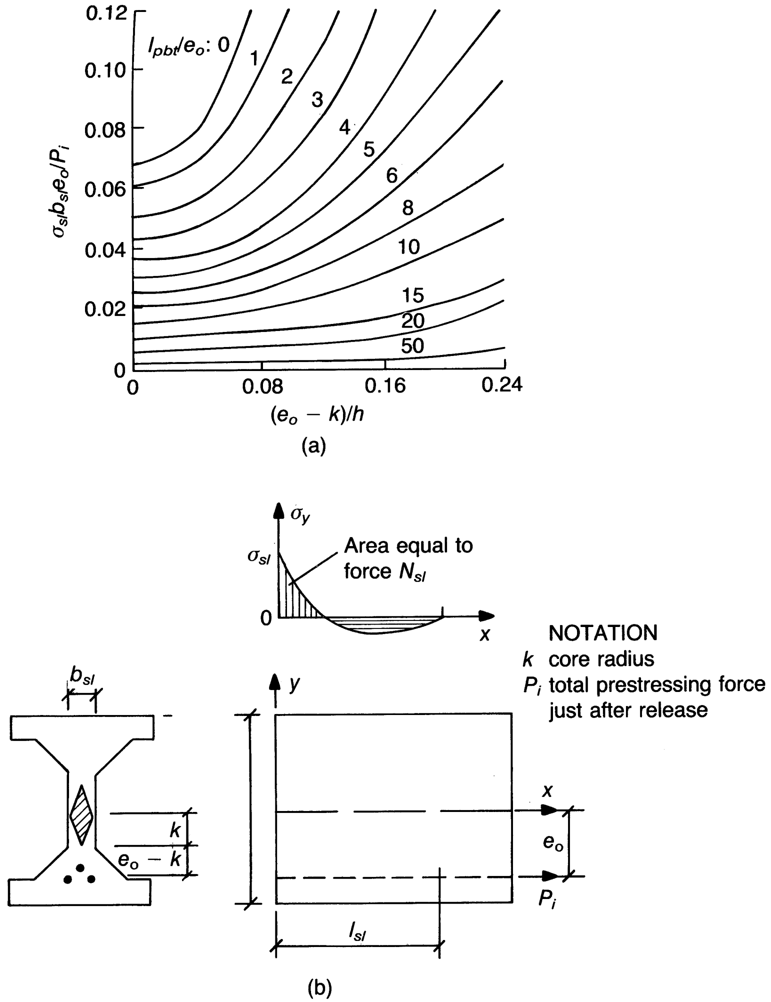


Fig. 6.9.13. Maximum spalling stress as a function of eccentricity and transmission length (based on linear analysis) for members with h < *400 mm*

6.9.12.4. Splitting

Splitting stresses due to bond of pretensioned tendons are sufficiently accounted for when the transverse reinforcement required for bursting and spalling confines the tendons.

If no such confining reinforcement is applied, the concrete cover should be as given in Table 6.9.3.

Table 6.9.3. Minimum cover as a function of the clear spacing to resist the splitting stresses around pretensioned tendons

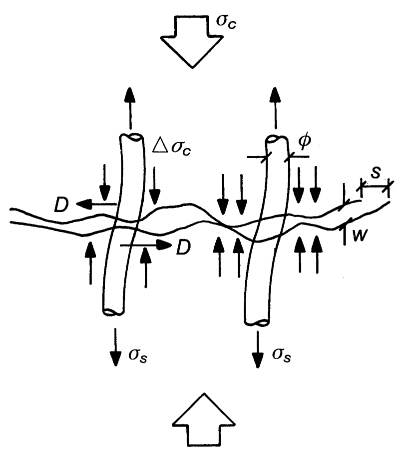
Clear spacing	Cover
$\geqslant 3\phi$	$\geqslant 3\phi$
$< 3\phi$	$\geqslant 4\phi$

6.10. ULS OF SHEAR JOINTS

6.10.1. General

This section covers design of reinforced concrete interfaces along which shear forces have to be transferred (cracked areas, joints between precast elements, etc.).

For a given value of relative displacement, the shear force which can be transferred along a reinforced concrete interface may be calculated on the basis of the basic models given in sections 3.9 (Concrete-to-concrete friction) and 3.10 (Dowel action).



Any *shear slip imposed* on an *interface* results in a dilatancy, w, whose value is a function of the geometry (natural rough, smooth, keyed) of the interface.

This dilatancy induces axial stresses in the reinforcing bars crossing the interface. These axial stresses may be calculated according to section 6.9.

The tensioned bars impose normal compressive stresses, $\Delta\sigma_c$, on the interface (clamping effect). Thus, the interface under the normal stress σ_c (due to external actions or prestressing) increased by $\Delta\sigma_c$ resists the imposed shear slip by means of friction. The friction resistance may be evaluated on the basis of section 3.9.

The reinforcing bars themselves contribute to the shear resistance of the joint by means of their dowel action (to be evaluated according to section 3.10).

The shear resistance of the joint, for a given slip value, may be calculated as the sum of the contributions from all resisting mechanisms.

If the construction is carried out according to the approximate rules, with particular regard to surface treatment, concrete compaction and plasticity, two categories of surface roughness are considered:

- Category 1 ('smooth')

I a smooth surface, as obtained by casting against a steel or timber shutter
II a surface which lies between trowelled or floated to a degree, which is effectively as smooth as (I)
III a surface which has been trowelled or tamped in such a way that small ridges, indentations or undulations have been left
IV a surface achieved by slip forming or vibro-beam screeding
V a surface achieved by extrusion
VI a surface, which has been deliberately textured by lightly brushing the concrete when wet

- Category 2 ('rough')

VII as for (VI), but with more pronounced texturing, as obtained by brushing, by a transverse screeder, by combining with a steel rake or with an expanded metal

6.10.2. Design of shear joints

Where a more general model is not available and SLS aspects are not governing, the following expression may be used for the calculation of the shear resistance of joints

$$\tau_{Rd} = \beta f_{ctd} + \mu(\rho f_{yd} + \sigma_{cd}) < 0.25 f_{cd} \qquad (6.10\text{–}1)$$

where

βf_{ctd} is the cohesion between the two concrete parts of the joints,
μ is the coefficient of friction,

the factors β and μ depend on the roughness-category of the interfaces, according to Table 6.10.1,

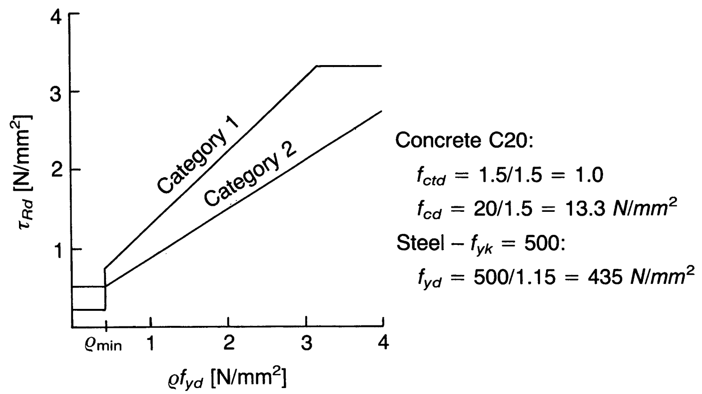


Fig. 6.10.2. Example of application

VIII a surface which has been thoroughly compacted, but no attempt has been made to smooth, tamp or texture the surface in any way, having a rough surface with coarse aggregate protruding, but firmly fixed in the matrix

IX where the concrete has been sprayed when wet, to expose the coarse aggregate without disturbing it

X a surface which has been provided with mechanical shear keys.

Eq. (6.10–1) applied for category 2, is an approximate linearization of eq. (3.9–3).

Table 6.10.1. Factors β and μ used in eq. (6.10–1)

	Surface category 1	Surface category 2
β	0.2*	0.4
μ	0.6	0.9

*For very smooth surfaces I and II (see commentary on left-hand side) the use of $\beta = 0.1$ is recommended.

ρ is the ratio of reinforcing steel crossing the joint $\geqslant 0.001$
f_{yd} is the design value of the yield stress of the steel
σ_{cd} is the external normal stress acting on the joint ($\sigma_{cd} > 0$ for compression)
f_{ctd} is the design tensile strength of concrete with lowest strength ($f_{ctk,\min}/1.50$).

For low shear situations, the design shear strength may be estimated according to

$$\tau_{Rd} = \beta f_{ctd}$$

In this case no reinforcement is necessary.

For the definition of shear joint geometry, and the role of an inclined reinforcement, reference is made to subsection 14.3.3.

7. VERIFICATION OF SERVICEABILITY LIMIT STATES

7.1. REQUIREMENTS

It should be demonstrated that the structure and the structural elements perform adequately in normal use. To meet this requirement the serviceability limit states should be verified.

Depending on the type and function of a structure or a structural element the verification of different serviceability limit states may be relevant, such as the limitation of

- stresses (see section 7.3)
- crack widths (see section 7.4)
- deformations (see section 7.5)
- vibrations (see section 7.6).

The serviceability limit states are listed in subsection 1.2.3.

The verification of SLS is performed under service load conditions and the operational failure probability to exceed the limit state is about a thousand times higher than that of ULS. Obviously it can be observed sporadically, that the requirements are not met. Such a situation should not justify rejection of the structure.

Exceeding the limit state of stresses or limit state of cracking may lead to limited local structural damage mainly affecting the durability of the structure.

Excessive deformations may produce damage in non-structural elements or load bearing walls and affect the efficient use or appearance of structural or non-structural elements.

Vibrations may cause discomfort, alarm or loss of ability to use.

7.2. DESIGN CRITERIA

For the verification of serviceability limit states all direct and indirect actions (loads or imposed or restrained deformations) should be taken into account.

The design criteria depend on the type of SLS and are given in clause 1.6.6.2.

The partial safety coefficients are taken equal to 1.0.

The combination of loads to be considered depends on the type of SLS and on the specific problem. It is suitable to utilize one of the combinations given in clause 1.6.6.5, i.e.

- rare combination
- frequent combination
- quasi-permanent combination.

For special problems other suitable combinations may be agreed by client and designer.

Prestressing forces should be considered as permanent actions.

The relevant values of the prestressing force depend on the type of SLS and the problem considered. Prestressing force values to be considered are suggested in section 4.6.

In general for LS of deformations the mean value is sufficient while for LS of cracking or stresses upper or lower fractiles are suitable.

For structural analysis any appropriate method may be used, which takes account of the material behaviour under service loads.

Linear or non-linear methods may be used. For most SLS problems linear analysis is sufficient. If, however, a non-linear analysis is carried out for ULS the action effects under service loads may be calculated by the same model. Plastic analysis is unsuitable for SLS.

7.3. STRESS LIMITATION

Under service load conditions the limitation of stresses may be required for

- tensile stresses in concrete
- compressive stresses in concrete
- tensile stresses in steel.

In selecting appropriate stress limits, the effect of the absolute dimensions of the member should be taken into account. Lower limits will be appropriate for larger members due to size-effects.

The limitation of tensile stresses in concrete is an adequate measure to reduce the probability of cracking.

For more detailed information see section 7.4.

The limitation of compressive stresses in concrete should avoid excessive compression, producing irreversible strains and longitudinal cracks.

Tensile stresses in reinforcement should be limited with an appropriate safety margin below the yielding strength, preventing uncontrolled cracking.

In calculating the stress, account shall be taken of whether the section is expected to crack under service loads and also of the effects of creep, shrinkage and relaxation of prestressing steel. Other indirect actions which could influence the stress, such as temperature, should also be considered.

Stresses are calculated using section properties corresponding to either the uncracked or the fully cracked condition, whichever is appropriate.

In general where the maximum tensile stress in the concrete calculated on the basis of an uncracked section under the rare combination of loads exceeds f_{ctm} (see Table 2.1.2), the cracked state should be assumed.

Where an uncracked section is used, the whole of the concrete section is assumed to be active and both concrete and steel are assumed to be elastic in both tension and compression. Where a cracked section is used, the concrete is assumed to be elastic in compression but to be incapable of sustaining any tension (in verifying stresses in accordance with these results, no allowance should be made for the stiffening effect of the concrete in tension after cracking).

At least the minimum area of reinforcement given by subsection 7.4.5 is required to satisfy the limitation of the stress in ordinary bonded reinforcement.

7.3.1. Tensile stresses in concrete

Depending on the stress limit chosen different limit states may be required, but the LS of decompression is considered to be the most significant. Stresses may be calculated on the basis of a homogeneous uncracked concrete section (state I). The contribution of reinforcement to the area and section modulus of the cross-section may be taken into account.

In specific cases, e.g. in incremental launching with precast elements, a minimum compressive stress may be required.

7.3.1.1. Limit state of decompression

The limit state of decompression is defined as the state where all axial concrete stresses are below or equal to zero.

As a rule the limit state of decompression should be required, if cracking or reopening of cracks are to be avoided under a given load combination. The margin between zero stress and tensile strength may also be reserved for self-equilibrating stresses not under consideration.

7.3.2. Compressive stresses in concrete

Excessive compressive stress in the concrete under service load may lead to longitudinal cracks and high and hardly predictable creep, with serious consequences to prestress losses. When such effects are likely to occur, measures should be taken to limit the stresses to an appropriate level.

If the stress does not exceed $0.6f_{ck}(t)$

- under the rare combination, longitudinal cracking is unlikely to occur
- under the quasi-permanent combination, creep and the corresponding prestress losses can be correctly predicted.

If under the quasi-permanent combination the stress exceeds $0.4f_{ck}(t)$, the non-linear model shall be used for the assessment of creep (see clause 2.1.6.4.3(d)).

The occurrence of longitudinal cracks may lead to a reduction in durability. In the absence of other measures (such as an increase of concrete cover) it is recommended to limit the compressive stress for exposure classes 3 and 4 (see Table 1.5.1). However, no limitation in serviceability conditions is necessary for stresses under bearings and anchorages.

The limit of $0.6f_{ck}(t)$ is not sharp. Consequently, in the corresponding verification prestress may be represented by its mean value, and in transient situations where the magnitude of variable actions is small (especially at transfer in prestressed beams) the quasi-permanent combination may be substituted by the rare combination. On the other hand prestress and concrete strength should be introduced in the verification by their values at the time at which the maximum stresses are reached.

These measures should be envisaged for deformations if the span/effective depth ratio exceeds 85% of the value given in Table 7.5.2 for the case considered.

If creep is likely to significantly affect the functioning of the member considered (e.g. with regard to loss of prestress, deformation, validity of the structural analysis) an alternative measure would be a limitation of the stress to $0.4f_{ck}(t)$. However, the limitation may be taken as a value between $0.4f_{ck}(t)$ and $0.6f_{ck}(t)$ for verifications relating to a transient situation (e.g. during construction) depending on the duration of the loading.

7.3.3. Steel stresses

Tensile stresses in the steel under serviceability conditions which could lead to inelastic deformation of the steel shall be avoided as this will lead to large, permanently open, cracks.

Creep effects in a cracked cross-section may be taken into account by assuming a modular ratio of 15 for situations where more than 50% of the stress arises from quasi-permanent actions. Otherwise, they may be ignored.

This requirement will be met provided that, under the rare combination of loads, the tensile stress in ordinary reinforcement does not exceed $0.8f_{yk}$. Where the stress is due only to imposed deformations, a stress of $1.0f_{yk}$ will be acceptable.

The stress in prestressing tendons should not exceed $0.75f_{pyk}$ after allowance for losses (see clause 2.3.4.1).

7.3.4. Cases where stress calculation is not essential

The stress limitations given in subsections 7.3.2 and 7.3.3 above may generally be assumed to be satisfied without further calculations provided

(*a*) the design for ultimate limit state has been carried out in accordance with chapter 6
(*b*) the minimum reinforcement provisions of subsection 7.4.5 are satisfied
(*c*) detailing is carried out in accordance with chapter 9
(*d*) not more than 30% of redistribution has been carried out in the analysis for the ultimate limit state.

Stress verifications should be carried out for partially prestressed members because there may be fatigue problems.

As mentioned in subsection 7.3.2 it may be necessary to verify the compressive stresses in prestressed beams at transfer.

7.4. LIMIT STATE OF CRACKING

7.4.1. Requirements

It should be ensured that, with an adequate probability, cracks will not impair the serviceability and durability of the structure.

Cracks do not, per se, indicate a lack of serviceability or durability; in reinforced concrete structures, cracking may be inevitable due to tension, bending, shear, torsion (resulting from either direct loading or restraint of imposed deformations), without necessarily impairing serviceability or durability.

However, the following specific requirements should generally be respected.

Cracks may be due to other causes such as plastic shrinkage or chemical reactions accompanied by expansion of the hardened concrete. The avoidance and the control of the width of such cracks are not covered by this chapter; see chapter 8 and Appendix d, sections d.6.3. and d.12.4.

7.4.1.1. Function requirements

The function of the structure should not be harmed by the cracks formed.

This may be the case in reservoirs or external water-insulation structures.

Alternatively, cracks may be allowed to form without any control of their width or the probability of their formation may be reduced by special measures, such as the provision of joints (provided that this does not impair the functioning of the structure).

In relevant cases, nominal crack width limits may be agreed with the client, unless reference is made to more simplified design means.

However, due to the actual state-of-the-art and the highly probabilistic nature of the related phenomena, such nominal crack width values may only serve as means to apply the design criterion of subsection 7.4.2a, and can in no case be compared to actual crack widths measured in situ.

7.4.1.2. Durability

However, under some well defined conditions, crack formation in reinforced concrete does not necessarily increase the corrosion risk of normal reinforcing steel; provided that the characteristic crack width does not exceed an appropriately specified value w_{lim}.

A typical exception of this rule should be carefully considered. It is the case of frequent use of de-icing agents on top of tension zones of reinforced concrete elements.

7.4.1.3. Appearance of the structure

The appearance of the structure should not be unacceptable because of cracking.

The satisfaction of this requirement should exclusively be made by means of deemed to satisfy rules. If not otherwise specified, the design criteria presented in this chapter to cover durability requirements are considered as satisfying this requirement too.

7.4.1.4. Uncertainties

Uncertainties related to the actual local concrete tensile strength, as well as to unforeseen tensile stresses, should be appropriately covered in design and construction.

7.4.1.5. Further requirements

Further requirements for an appropriate control of cracking may result from the necessity to limit or to avoid

- vibrations
- damage caused by excessive deformations
- brittle failure.

Brittleness of structural elements due to crack formation is avoided, if the minimum reinforcement ratios required in chapter 9 are observed.

7.4.2. Design criteria vs. cracking

(*a*) The specific requirement of clauses 7.4.1.1 to 7.4.1.4 may be met by an appropriate limitation of crack widths. This may be achieved either by means of analytical procedures (clause 7.4.3.1 or 7.4.3.2) or by appropriate practical rules (subsection 7.4.4).

(*b*) In cases where the ULS design leads to low reinforcement ratios, the specific requirements of subsection 7.4.1 may be met by providing an appropriate minimum amount of reinforcement.

Thus, at all sections which are expected to be subjected to significant tension (due to restraint, combined or not with direct loading), a minimum amount of reinforcement should be placed, ensuring that yield of the reinforcement will not occur after cracking in the SLS.

This applies also to prestressed members in regions where tension is expected to develop in the concrete.

Different combinations of actions may be considered under particular conditions (e.g. partially prestressed structures under special conditions).

(*a*) In the absence of specific requirements (e.g. watertightness), it may be assumed that for exposure classes 2–4 (as specified in section 1.5), a $w_{\lim}$-value equal to 0.30 mm under the quasi-permanent combination of actions is satisfactory for reinforced concrete members with respect to both appearance and durability.

For exposure class 1, this limit may be relaxed provided that it is not necessary for reasons other than durability.

When de-icing agents are expected to be used on top of tensioned zones of reinforced concrete elements, appropriate $w_{\lim}$-values should be specified in accordance with the client, depending on the thickness and quality of the concrete, and of additional protective layers.

(*b*) For prestressed members, if more detailed data are not available, the crack width limiting values presented in Table 7.4.1 may be used.

Table 7.4.1. Crack width limits for prestressed members

Exposure class	Limiting crack widths (in mm) under the frequent load combination	
	Post-tensioned	Pretensioned
1	0.20	0.20
2	0.20	No tension within the section is allowed
3 and 4	(*a*) No tension is allowed within the section, or (*b*) if tension is accepted, impermeable ducts or coating of the tendons should be applied; in this case, $w_{\lim} = 0.20$	

7.4.3. Verification of crack width

(*a*) Whenever an analytical procedure is needed, the criterion of sub-section 7.4.2a is applied as follows for transverse cracks.
The following inequality should be observed

$$w_k \leqslant w_{\lim} \tag{7.4–1}$$

where

w_k denotes the characteristic crack width calculated as in clause 7.4.3.1 or 7.4.3.2 under the appropriate combination of actions (see clause 1.6.6.5)

$w_{\lim}$ denotes the nominal limit value of crack width which is specified for cases of expected functional consequences of cracking, or for some particular cases related to durability problems.

Longitudinal cracks due to the corrosion of steel bars are not covered by these criteria and shall be avoided by means of measures taken to ensure durability (see section 8.4).

If

$$\rho_{s,ef}\sigma_{s2} > f_{ctm}(t)(1 + \alpha_e \rho_{s,ef})$$

it may be assumed that the stabilized cracking condition has been reached, otherwise the formation of single cracks should be considered; where

- $f_{ctm}(t)$ is the mean value of the tensile strength of the concrete at the time t when the crack appeared,
- α_e is the ratio E_s/E_{ci} (for E_{ci} see clause 2.1.4.2; in the case of early cracking the modulus of elasticity should be reduced according to clause 2.1.6.3)
- $\rho_{s,ef}$ is the effective reinforcement ratio ($= A_s/A_{c,ef}$)
- $A_{c,ef}$ is the effective area of concrete in tension; this is generally the area of concrete surrounding the tension reinforcement (see Fig. 7.4.2)
- σ_{s2} denotes the steel stress at the crack.

For the sake of simplicity $(1 + \alpha_e \rho_{s,ef})$ can be set equal to 1.

$$l_{s,\max} = 2\,\frac{\sigma_{s2} - \sigma_{sE}}{4\tau_{bk}}\,\phi_s \qquad (7.4\text{–}3)$$

$$= \frac{\phi_s}{3.6\rho_{s,ef}} \text{ for stabilized cracking}$$

$$= \frac{\sigma_{s2}}{2\tau_{bk}}\,\phi_s\,\frac{1}{1 + \alpha_e \rho_{s,ef}} \text{ for single crack formation}$$

(*b*) For the control of longitudinal cracks (parallel to main steel bars), the following design criteria apply.

(i) The thickness of concrete cover as well as, where necessary, its secondary reinforcement (skin reinforcement) transverse to the main steel bars should be appropriately selected (as a function of their diameter), in order to secure the full development of bond resistance without any longitudinal cracking.

(ii) For plane elements reinforced in two directions, tensile stresses generated in sections parallel to the direction of a steel bar should be appropriately limited.

7.4.3.1. Calculation of crack width in reinforced concrete members

7.4.3.1.1. Basic crack width formula

For all stages of cracking, the design crack width may be calculated according to

$$w_k = l_{s,\max}(\varepsilon_{sm} - \varepsilon_{cm} - \varepsilon_{cs}) \qquad (7.4\text{–}2)$$

where

- $l_{s,\max}$ denotes the length over which slip between steel and concrete occurs; steel and concrete strains, which occur within this length, contribute to the width of the crack; $l_{s,\max}$ is calculated by means of eq. (7.4–3)
- ε_{sm} is the average steel strain within $l_{s,\max}$
- ε_{cm} is the average concrete strain within $l_{s,\max}$
- ε_{cs} denotes the strain of concrete due to shrinkage; it has to be introduced algebraically.

where

σ_{sE} is the steel stress at the point of zero slip
τ_{bk} is the lower fractile value of the average bond stress; it may be taken from Table 7.4.2
ϕ_s denotes the diameter of the steel bar, or the equivalent diameter of bundled bars.

From eq. (7.4–2) can be derived with Fig. 7.4.1 for the mean strains

$$\varepsilon_{sm} - \varepsilon_{cm} = (\varepsilon_{s2} - \beta\Delta\varepsilon_{sr}) - \beta\varepsilon_{sr1} = \varepsilon_{s2} - \beta\varepsilon_{sr2} \qquad (7.4\text{–}4)$$

with

$$\varepsilon_{sr2} = \frac{f_{ctm}(t)}{\rho_{s,ef} E_s}(1 + \alpha_e \rho_{s,ef})$$

where

$$\Delta\varepsilon_{sr} = \varepsilon_{sr2} - \varepsilon_{sr1}$$

ε_{s2} is the steel strain at the crack
ε_{sr2} is the steel strain at the crack, under forces causing $f_{ctm}(t)$ within $A_{c,ef}$; if the internal forces are lower than or equal to these forces (e.g. in a working joint), then $\varepsilon_{sr2} = \varepsilon_{s2}$
ε_{sr1} is the steel strain at the point of zero slip under cracking forces reaching $f_{ctm}(t)$
β is an empirical factor to assess averaged strain within $l_{s,\max}$; it can be taken from Table 7.4.2
$f_{ctm}(t)$ is the mean value of concrete tensile strength at time t at which the crack appeared.

For direct calculation of the reinforcement area A_s, required for crack width control with a given bar diameter, the following formula can be used

$$A_s = \sqrt{\left[\frac{\phi_s F_{cr}(F_s - \beta F_{cr})}{2E_s w_k \tau_{bk}(1 + \alpha_e \rho_{s,ef})}\right]} \qquad (7.4\text{–}5)$$

where

$(1 + \alpha_e \rho_{s,ef}) = 1$, is allowed for a simple calculation
F_s is the force in the crack transmitted by the reinforcement
F_{cr} indicates the force, which has to be introduced into concrete by bond (or interaction with other parts of the structure) in order to provoke cracking within $A_{c,ef}$ at the end of the transmission length

$$F_{cr} = F_s$$

for the crack formation phase $[\rho_{s,ef}\sigma_{s2} \leq f_{ctm}(t)(1 + \alpha_e \rho_{s,ef})]$

$F_{cr} = A_{c,ef} f_{ctm}(t)(1 + \alpha_e \rho_{s,ef})$ for stabilized cracking $[\rho_{s,ef}\sigma_{s2} > f_{ctm}(t)$ $(1 + \alpha_e \rho_{s,ef})]$.

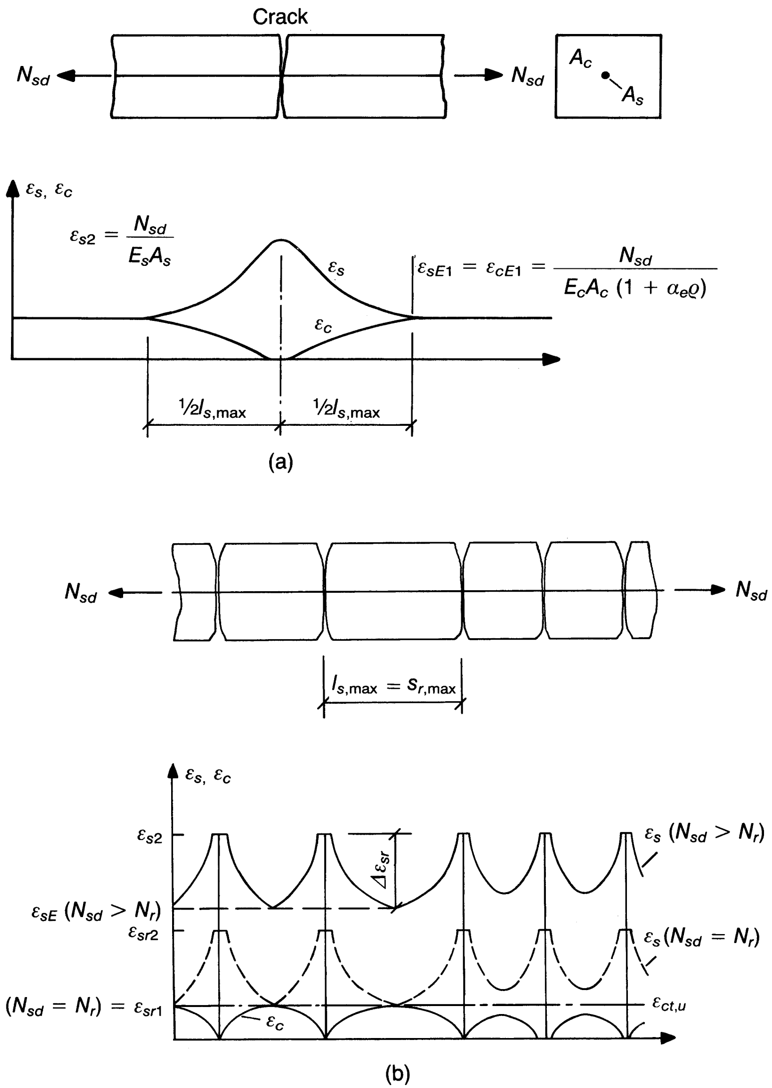


Fig. 7.4.1. Strains for calculating the crack spacing and the average strains: (a) for single cracks; (b) for stabilized cracking

For stabilized cracking the average width may be estimated on the basis of an average crack spacing: $s_{rm} \approx \frac{2}{3} l_{s,\max}$.

Table 7.4.2. Values for β and τ_{bk} (assuming that only deformed bars are used)

	Single crack formation		Stabilized cracking	
	β	τ_{bk}	β	τ_{bk}
Short term/instantaneous loading	0.6	$1.8 f_{ctm}(t)$	0.6	$1.8 f_{ctm}(t)$
Long term/repeated loading	0.6	$1.35 f_{ctm}(t)$	0.38	$1.8 f_{ctm}(t)$

The effective concrete area in tension ($A_{c,\mathrm{ef}}$) accounts for the non-uniform normal stress distribution by bond forces into the concrete cross-section at the end of the transmission length.

In absence of a refined model, Fig. 7.4.2 may be used in order to assess the effective concrete area in tension.

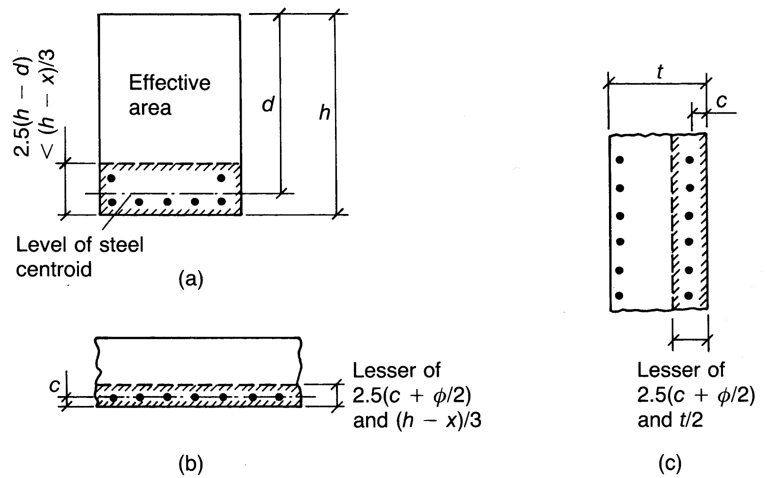


Fig. 7.4.2. Effective tension area: (a) beam; (b) slab; (c) member in tension

By means of the method given in this section, the design crack width within the effective tension area may be calculated. It should be noted that outside this region, larger cracks may occur.

When a more refined model is not available, the following expression may be used:

$$l_{s,\max} = \left(\frac{\cos\theta}{l_{sx,\max}} + \frac{\sin\theta}{l_{sy,\max}}\right)^{-1} \tag{7.4–7}$$

where

θ denotes the angle between the reinforcement in the x-direction and the direction of the principal tensile stress

$l_{sx,\max}$, $l_{sy,\max}$ denote crack spacings in the two orthogonal directions, calculated according to eq. (7.4–3).

The crack spacing calculated taking into account prestressing tendons is valid for a square region (300 mm × 300 mm) surrounding the prestressing tendon.

For single cracks $(\Delta F_{s+p} \leqslant f_{ctm}(t) A_{c,ef})$

For the sake of simplicity the factor $(1 + \alpha_e \rho)$ is taken as 1 in the whole clause.

For reinforcing steel $l_{s,\max}/2 = \sigma_{s2}\phi_s/(4\tau_{bs,k})$.

For prestressing steel $l_{p,\max}/2 = \Delta\sigma_p\phi_p/(4\tau_{bp,k})$.

7.4.3.1.2. Combined effects of load and imposed deformations

Where cracking is due also to imposed deformations, the steel strain at cracks due to imposed loads (ε_{s2} in eq. (7.4–4)) should be increased by that caused by imposed deformations.

7.4.3.1.3. Orthogonal reinforcement directions

When in a member reinforced in two orthogonal directions, cracks are expected to form at a significant angle (> 15°) with respect to the direction of reinforcement, approximations may be used in calculating crack spacings.

7.4.3.2. Calculation of crack width for prestressed concrete

The calculation of crack width concerns structural members with bonded prestressing reinforcement.

The calculation of crack width follows generally the procedure and the formulae given in clause 7.4.3.1.

When prestressed and non-prestressed types of steel are simultaneously used, since the bond behaviour of prestressing tendons is different from the bond behaviour of deformed reinforcing bars, different steel stresses will be developed in each type of steel.

Both equilibrium and compatibility should be respected for the calculated stresses in prestressing and reinforcing steel.

For single cracks, different transmission lengths l_s and l_p should be calculated for reinforcing and prestressing steel respectively.

Assuming that $w_{sk} = w_{pk}$ according to eq. (7.4–2), the total force after decompression

$$\Delta F_{s+p} = A_s \sigma_{s2} + A_p \Delta\sigma_p$$

leads to different stresses in the reinforcing and prestressing steel

$$\sigma_{s2} = \Delta F_{s+p}/(A_s + \sqrt{(\xi_1)}A_p) \quad \text{in the reinforcing steel} \quad (7.4\text{–}8)$$

$$\Delta\sigma_P = \sqrt{(\xi_1)}\Delta F_{s+p}/(A_s + \sqrt{(\xi_1)}A_p) \text{ in the prestressing steel} \quad (7.4\text{–}9)$$

where

$$\xi_1 = \tau_{bp,k}\phi_s/(\tau_{bs,k}\phi_p)$$

$l_{s,\max}$ and $\varepsilon_{s2} = \sigma_{s2}/E_s$ can be introduced in eqs (7.4–2) and (7.4–4) in order to determine explicitly the crack width.

The force ΔF_{s+p} can be taken as follows.

— For tension chords of T-beams or box girders

$$\Delta F_{s+p} = 0.9f_{t1} \quad (7.4\text{–}10a)$$

— For rectangular sections or for the part of the web, where no tension chord is connected:

$$\Delta F_{s+p} = 0.9F_{t1}\left(1 - \frac{\sigma_{cs}}{\sigma^*_{cs}}\right) \quad (7.4\text{–}10b)$$

where

F_{t1} is the tensile force within the total tensile zone before formation of the first crack

σ_{cs} is the compressive stress at the centre of the section, due to external normal force N caused by load or restraint and the characteristic prestressing force including the losses due to creep and shrinkage

σ^*_{cs} is the compressive stress at the centre of gravity of the section which is able to control the crack width without any additional reinforcement in the tensile zone. This condition is fulfilled, if the depth of the tension zone, calculated on the basis of a cracked section under the loading conditions leading to formation of the first crack, does not exceed the lesser of $h/4$ or 0.3 m.

For stabilized cracking $(\Delta F_{s+p} > f_{ctm}(t) A_{c,ef})$

$$l_{s,\max} = l_{p,\max} = \frac{\phi_s}{3.6(\rho_{s,ef} + \xi_1 \rho_{p,ef})} \tag{7.4–11}$$

$$\sigma_{s2} = \sigma_{sm,m} + \tfrac{2}{3}\beta \frac{f_{ctm}(t)}{(\rho_{s,ef} + \xi_1 \rho_{p,ef})} \tag{7.4–12}$$

$$\Delta\sigma_p = \Delta\sigma_{pm,m} + \tfrac{2}{3}\beta \frac{\xi_1 f_{ctm}(t)}{(\rho_{s,ef} + \xi_1 \rho_{p,ef})} \tag{7.4–13}$$

where the subscript *ef* implies that steel areas should be normalized to appropriate effective concrete area in tension, and

σ_{s2} denotes the stress in the reinforcing steel at the crack, calculated on the basis of different bond characteristics for reinforcing bars and prestressing tendons

$\rho_{p,ef} = A_p / A_{c,ef}$
$\rho_{s,ef} = A_s / A_{c,ef}$
$\sigma_{sm,m} = \Delta\sigma_{pm,m}$ are the mean steel stresses; they can be calculated according to state II but taking the tension stiffening effect into account; the higher the percentage of steel the more the mean steel stress reach the steel stress according to the 'naked' state II

$l_{s,\max}$ and $\varepsilon_{s2} = \sigma_{s2}/E_s$ can be introduced in eqs (7.4–2) and (7.4–4) in order to determine explicitly the crack widths.

When the maximum slip between steel and concrete does not exceed the value of 0.25 mm (i.e. for $w < 0.50$ mm), the fractile value of the average bond stress for ribbed bars is given by

$$\tau_{bs,k} = k_1 f_{ctm}(t)$$

where

$k_1 = 2.25$ leading to 50% fractile
$k_1 = 1.80$ leading to 75% fractile of crack width for ribbed bars.

For smooth bars, 50% of the values of the average bond stress for ribbed bars should be taken. For prestressing steel the following apply

$\tau_{bp,k}/\tau_{bs,k} = 0.20$ for post-tensioned tendons, smooth bars
$\tau_{bp,k}/\tau_{bs,k} = 0.40$ for post-tensioned tendons, indented wires or strands
$\tau_{bp,k}/\tau_{bs,k} = 0.60$ for post-tensioned tendons, ribbed bars

In case of stabilized cracking, the maximum crack spacing may be calculated as in clause 7.4.3.1, provided that the different diameters and bond behaviour of reinforcing and prestressing steel are appropriately taken into account.

$\tau_{bp,k}/\tau_{bs,k}$ = 0.80 for pretensioned tendons, ribbed bars
$\tau_{bp,k}/\tau_{bs,k}$ = 0.60 for pretensioned tendons, indented bars or strands.

For reinforced or prestressed slabs subjected to bending without significant axial tension, no special measures to control cracking are needed, provided that the overall depth of the slab does not exceed 160 mm.

When the values given in the following Tables 7.4.3 and 7.4.4 are respected, crack widths do not generally exceed the value of 0.30 mm for reinforced elements and 0.20 mm for prestressed elements.

(*a*) For cracking caused mainly by restraint, crack widths will not generally be excessive provided that the bar sizes given in Table 7.4.3 are not exceeded; the σ_s-value of Table 7.4.3 is that calculated at cracking of the element (σ_{sr}).

(*b*) For cracks caused mainly by imposed loads, crack widths will not generally be excessive provided that either the provisions of Table 7.4.3 or those of Table 7.4.4 are satisfied.

Table 7.4.3. Maximum bar diameters (deformed bars) for which no calculation of crack width is needed

Steel stress (MPa)*	Maximum bar diameter (mm)	
	Reinforced sections	Prestressed sections
160	32	25
200	25	16
240	20	12
280	14	8
320	10	6
360	8	5
400	6	4
450	5	—

* Steel stresses are calculated under quasi-permanent loads (reinforced concrete) or under frequent loads and the characteristic value of prestress (prestressed concrete).

7.4.4. Control of cracking without calculation of crack width

Under well specified conditions, the fulfilment of the requirements of clauses 7.4.1.1, 7.4.1.2 and 7.4.1.3 may also be achieved by means of appropriate practical rules.

(*a*) When small depth elements subjected mainly to bending are considered, no special measures are needed for crack control.

(*b*) Under the condition that the minimum reinforcement specified in subsection 7.4.5 is provided, the design crack width may be kept to acceptable low values, if appropriately chosen bar diameters and bar spacings are used.

Further guidance concerning the choice of bar diameters is given in chapter 8.

For prestressed concrete sections, the stresses in the reinforcement should be calculated regarding prestress as an external force. In general the stress increase of the tendons, i.e. the contribution of tendons to the limitation of crack widths, may be disregarded.

For reinforced concrete the maximum bar diameter may be modified as follows.

For restraint cracking

$$\phi = \phi_{s,\max} \frac{f_{ctm}}{2.9} \qquad (7.4\text{–}14)$$

For load induced cracking

$$\phi_s = \phi_{s,\max} \frac{h_t}{7.5(h - d)} > \phi_{s,\max} \qquad (7.4\text{–}15)$$

where

ϕ_s is the adjusted maximum bar diameter
$\phi_{s,\max}$ is the maximum bar size given in the table
h is the overall depth of the section
h_t is the depth of the tension zone just before cracking
d is the effective depth
$f_{ctm}(t)$ is the mean value of the concrete tensile strength, at the time t when the crack appeared.

Table 7.4.4. Maximum bar spacing for which no calculation of crack width is needed

Steel stress (MPa)	Maximum bar spacing (mm)*	
	Reinforced sections	Prestressed sections
160	300	200
200	250	150
240	200	100
280	150	50
320	100	—
360	60	—

* For members in pure tension with an overall depth $h \leqslant 200$ mm and in pure bending with $h \leqslant 400$ mm a more accurate calculation could give greater spacings, but they should not exceed 300 mm.

7.4.5. Minimum reinforcement areas

7.4.5.1. General

A minimum amount of reinforcement should be provided in order to satisfy the requirements of clauses 7.4.1.1, 7.4.1.2 and 7.4.1.3.

To this end, in every area where (under SLS conditions) the tensile strength of concrete may be exceeded, an appropriate amount of reinforcement, for crack control, should be provided.

A combination of external loads and restraint or imposed deformations (intrinsic, like shrinkage or extrinsic, like differential settlements) may lead to this situation.

7.4.5.2. Mechanical basis

In calculating the minimum amount of reinforcement, redistribution of internal stresses after cracking, as well as fracture mechanics effects may be taken into account.

7.4.5.3. Simplified methods

Simplified calculation methods based on experimental evidence may be used.

For the combination of pure tension and flexure, in the absence of a more rigorous method, the following simplified procedure may be applied for the calculation of the required area of minimum reinforcement within the tensioned concrete zone

$$A_{s,\min} = k_c k f_{ct,\max} A_{ct} / \sigma_{s2} \qquad (7.4\text{–}16)$$

where

A_{ct} denotes the area of the concrete tension zone just before the formation of cracks, calculated with the technical theory in the uncracked stage

σ_{s2} may be taken equal to f_{yk} if adequate anchorage is secured; a lower value may, however, be needed to satisfy the crack width limits (see Tables 7.4.3 and 7.4.4)

$f_{ct,\max}$ is the upper fractile of the concrete strength in tension at the moment when the first crack is expected to appear; values of $f_{ct,max}$ may be obtained from Table 2.1.2

k is a factor correcting the value A_s as found by technical theory, against the real A_{ct}-values taking into account non-linear stress distributions (self-equilibrating effects); the following rules may be applied

- restraint of extrinsic imposed deformations

$$k = 1.0$$

- restraint of intrinsic imposed deformations of rectangular sections

$$k = 0.8 \text{ for } h < 0.3\,\text{m}$$
$$k = 0.5 \text{ for } h > 0.8\,\text{m}$$

(linear interpolation is possible)

k_c accounts for the scheme of tensile stress distribution

$k_c = 1.0$ for pure tension
$k_c = 0.4$ under flexural conditions without axial compressive force
$k_c = 0.4$–1.0 for a combination of pure tension and flexure.

In prestressed members, the minimum reinforcement for crack control is not necessary in areas where, under the rare combination of loads and the characteristic value of prestress or normal force, the concrete remains in compression.

Otherwise, the required minimum area may be calculated by means of equation (7.4–16), with the following values for k_c.

For box sections

$k_c = 0.45$ for the webs
$k_c = 0.9$ for the tension chord.

7.4.5.4. Reduced minimum reinforcement

In prestressed members or reinforced concrete members subject to compressive normal force, the minimum reinforcement area may be reduced below that necessary for ordinary reinforced concrete due to the influence of

- the increased flexural stiffness of the compression zone
- the contribution of the prestressing tendons
- the effect of prestress or compressive normal force contributing to crack width limitation of single cracks.

The minimum reinforcement may be reduced or even dispensed with

For rectangular sections

k_c = 0.45 under flexural conditions without axial compressive force

k_c = 0 when concrete remains in compression, or the depth of the tension zone, calculated on the basis of a cracked section under the loading conditions leading to formation of the first crack, does not exceed the lesser of $h/4$ or 0.3 m.

Linear interpolation between both values is possible.

Prestressing tendons may be taken into account as minimum reinforcement within a 300 mm square surrounding the tendon, provided the different bond behaviour of the tendons and reinforcement are taken into account.

altogether if the imposed deformation is so small that it is unlikely to cause cracking. In such cases minimum reinforcement is only needed to resist the forces due to restraint.

7.5. LIMIT STATES OF DEFORMATION

7.5.1. General

7.5.1.1. Requirements

In-service deformations (deflections and rotations) may be harmful to

- the appearance of the structure
- the integrity of non-structural parts
- the proper function of the structure or its equipment.

To avoid harmful effects of deformations appropriate limiting values should be respected.

To establish such limits is not within the scope of this Model Code. However, some practical rules are given in clause 7.5.2.3 for some categories of simple buildings.

Where applicable, acceptable limit values should be established in agreement with the client or his representative.

7.5.1.2. Combination of actions

The combinations of actions to be considered depend on the criteria in question and are defined in section 7.2.

In order to ensure a satisfactory behaviour in the serviceability limit state, deformations should be calculated as follows

- the long-term deformations are calculated for the quasi-permanent combinations
- the instantaneous deformations should be calculated for the rare combinations.

For the calculation of camber, only the quasi-permanent combinations are considered.

7.5.1.3. Data for the materials

The values of the material properties to be applied depend on the criteria in question.

In order to prevent damage due to deformations, prudent values of the material properties should be used.

In order to calculate camber, the mean values of the material properties may be used.

7.5.1.4. Modelling

Depending on the precision needed, appropriate deformation models should be used, as described in the following subsections.

7.5.2. Deformations due to bending with or without axial force

7.5.2.1. General methods

The deformations are calculated from the curvatures (see section 3.6) by applying appropriate procedures, such as the principle of virtual work or double integration.

In state I the assumption of plane sections remaining plane is accepted. The principle of superposition and, thus, linearity are assumed valid.

In state II-naked the assumption of plane sections is applied.

For quasi-permanent load combinations, the time-dependent behaviour of the materials should be taken into account.

The actual deformations may differ appreciably from the calculated values; in particular if the values of the applied moments are close to the cracking moment. The difference will depend on the dispersion of the material properties, on the ambient conditions, on the loading conditions and the previous loading conditions, on the restraints at the supports, etc.

For prestressed concrete it may be necessary to control deflections assuming unfavourable deviations of the prestressing force and the dead load.

General methods are proposed in CEB Bulletin d'Information No. 158, CEB-Manual Cracking and Deformations, Lausanne, 1985.

7.5.2.2. Simplified method

For building members, long-term deflections can be evaluated by the following relations based on a bilinear relationship between load and deflection

$$a = (1 + \phi)a_c \quad \text{for} \quad M_d < M_r \tag{7.5–1a}$$

$$a = \left(\frac{h}{d}\right)^3 \eta(1 - 20\rho_{cm})a_c \quad \text{for} \quad M_d \geqslant M_r \tag{7.5–1b}$$

Equation (7.5–1b) is only valid for reinforced concrete members.

with cracking moment $M_r = W_c f_{ct}$

where

a_c is the elastic deflection calculated with the rigidity $E_c I_c$ of the cross section (neglecting the reinforcement)

M_d is the bending moment at mid-span of a beam or a slab, or at the fixed end of a cantilever under frequent actions
ρ_{tm} is the geometrical mean percentage of tensile reinforcement, see eq. (7.5–2)
ρ_{cm} is the geometrical mean percentage of compressive reinforcement
η is a correction factor (see Table 7.5.1), which includes the effects of cracking and creep
ϕ is the creep coefficient (see clause 2.1.6.4.3).

Table 7.5.1. Correction factor η for estimation of deflections

$\rho_m(\%)$	0.15	0.2	0.3	0.5	0.75	1.0	1.5
η	10	8	6	4	3	2.5	2

The mean percentage ρ_m of tensile reinforcement is determined according to the bending moment diagram (see Fig. 7.5.1).

$$\rho_m = \rho_a \frac{l_a}{l} + \rho \frac{l_o}{l} + \rho_b \frac{l_b}{l} \qquad (7.5\text{–}2)$$

where

ρ_a, ρ_b are the percentages of tensile/compressive reinforcement at the left and right supports, respectively,
ρ is the percentage of tensile reinforcement at the $M_{\max}$-section.

An estimate of the lengths l_a, l_b is generally sufficient.

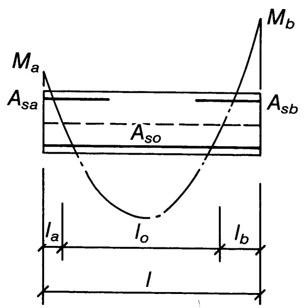


Fig. 7.5.1. Bending moment diagram defining l_o, l_a, l_b

7.5.2.3. Practical verification of deflections

For simple building elements under specified circumstances it may not be necessary to calculate deflections explicitly if certain limitations of the span-depth ratio are respected.

Making use of the simplified provisions of clause 7.5.2.2, the following rule may be applied

$$l/d \leqslant 1/\sqrt{[\delta\eta(l/a)_{\lim}]^{1/3}} \qquad (7.5\text{–}3)$$

where δ is a coefficient characterizing the system

For the derivation of such limiting values, the following criteria should be fulfilled

- the deformation sensitivity of non-structural elements attached to (or in contact with) the flexural member under consideration, should be accounted for
- simple but sound models for deflection estimation should be used
- previous experience within the prescribed fields of application should be available.

For simple verifications $(l/a)_{\lim} = 300$.

$(l/a)_{\lim}$ is an appropriate span-deflection limiting value.

When applying eqs (7.5–3) and (7.5–4), it is allowable to replace any shape of a cross-section by a rectangular one with the same height and the same moment of inertia. The cracking moment M_r is calculated for the original cross-section.

For two-way spanning slabs, the verification should be carried out for the shorter span. For flat slabs the longer span should be taken.

For reinforced concrete flexural elements without axial force the following rule may be applied

$$\frac{l}{d} \leqslant \lambda = \lambda_o k_T k_l \left(\frac{400}{f_{yk}}\right) \qquad (7.5\text{–}5)$$

where

λ_o is taken from Table 7.5.2
$k_T = 1.0$ for flanged section with flange-web width ratio lower than 3 and 0.8 for ratios higher than 3
$k_l = 7/l \leqslant 1$, with l in m
f_{yk} is the yield stress of the reinforcing steel (MPa).

Members where the concrete is lightly stressed are those where $\rho < 0.5\%$ ($\rho = A_s/bd$). It may normally be assumed that slabs are lightly stressed.

If the reinforcement ratio ρ is known, (λ_o)-values between the 'highly stressed' and 'lightly stressed' values in Table 7.5.2 may be obtained by interpolation, assuming the 'lightly stressed' values to correspond to $\rho = 0.5\%$ and the 'highly stressed' to $\rho = 1.5\%$.

Table 7.5.2. Values of λ_o for reinforced concrete members without axial compression

Structural system	Concrete highly stressed	Concrete lightly stressed
Simply supported beam, one- or two-way spanning simply supported slab	18	25
End span of a series of continuous spans, two-way spanning slab continuous over one long side	23	32
Interior span of beam or one-way or two-way spanning slab	25	35
Slab supported on columns without beams (flat slab), verification effected on the longer span	21*	30*
Cantilever	7	10

* These values should be verified.

The limits given for flat slabs correspond to a less severe limitation than a mid-span deflection $l/250$. Experience has shown this to be satisfactory.

The mean strain ε (instantaneous or long-term) in any section of a tie is defined, for the first loading, as follows

$$\varepsilon = \varepsilon_1 \qquad \text{for } N < N_r \tag{7.5–6a}$$

$$\varepsilon = \varepsilon_2 - (\varepsilon_{2r} - \varepsilon_{1r})\beta_t \qquad \text{for } N \geqslant N_r \tag{7.5–6b}$$

where

N is the applied normal force on the section
N_r is the cracking normal force

$$N_r = f_{ct}A_1 \tag{7.5–7}$$

where

f_{ct} is the tensile strength
A_1 is the section area in state I
ε_1, ε_{1r} are the strains in state I corresponding to the actions N and N_r respectively
ε_2, ε_{2r} are the strains in state II-naked corresponding to the actions N and N_r respectively
$\beta_t = 0.40$ for instantaneous loads and 0.25 for long-term or repeated loads.

7.5.3. Other deformations

7.5.3.1. Deformations due to pure tension

The deformations due to pure tension may be calculated under service conditions when the tension stiffening is assumed constant (see section 3.2).

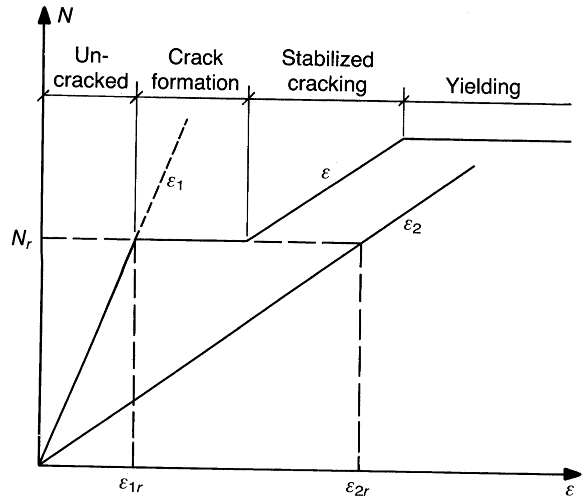


Fig. 7.5.2. Instantaneous mean strain

7.5.3.2. Deformations due to shear forces

If diagonal cracking does not occur, deformations due to shear forces may be neglected.

Influences on shear stiffness from normal force or prestress may generally be neglected.

7.5.3.3. Deformation due to torsion

This clause applies to linear elements of hollow or solid cross-section, subjected to pure or combined torsion.

The torsional stiffness should be determined taking account of cracking under the relevant load effects.

The torsional stiffness can be determined for the compressed part of the

The shear modulus can be taken as

$$G = 0.4E_{cm}$$

E_{cm} is the mean secant modulus in the compression zone using the stress–strain relation for concrete in compression.

For simplification the shear centre can be determined by using the uncracked cross-section.

7.6. VIBRATIONS

7.6.1. General

Vibrations of structures may affect the serviceability of a structure as follows

- functional effects (discomfort to occupants, affecting operation of machines, etc.),
- structural effects (mostly on non-structural elements, as cracks in partition, loss of cladding, etc.).

Vibrations can be caused by several variable actions, e.g.

- rhythmic movements made by people such as walking, running, jumping and dancing
- machines
- waves due to wind and water
- rail and road traffic
- construction work such as driving or placing by vibration of sheet piles, compressing soil by means of vibrations as well as blasting work.

Vibrations that endanger the structure, such as very large deflections due to resonance or the loss of resistance due to fatigue, should be included in the verification for ULS of the structure.

7.6.2. Vibrational behaviour

To secure satisfactory behaviour of a structure subject to vibrations, the natural frequency of vibration of the relevant structure should be kept sufficiently apart from critical values which depend on the function of the corresponding building, see Table 7.6.1.

$$f > kf_{crit} \quad \text{or} \quad f < f_{crit}/k$$

where k takes integer values.

The vibrational behaviour of structures can be influenced by the following measures

- changing the dynamic actions
- changing the natural frequencies by changing the rigidity of the structure or the vibrating mass
- increasing the damping features, etc.

Table 7.6.1. Critical frequency in structures subject to vibrations caused by movements of people

Structures	Frequency (Hz) f_{crit}
Gymnasia and sports halls	8.0
Dance rooms and concert halls without permanent seating	7.0
Concert halls with permanent seating	3.4
Structures for pedestrians and cyclists	See below*

* Natural frequencies between 1.6 and 2.4 Hz and between 3.5 and 4.5 Hz are to be avoided in structures for pedestrians and cyclists. Joggers can also cause vibrations in structures with natural frequencies between 2.4 and 3.5 Hz.

8. DURABILITY

8.1. GENERAL

The basic requirement for design versus durability (subsection 1.5.1) is

> Concrete structures shall be designed, constructed and operated in such a way that, under the expected environmental influences, they maintain their safety, serviceability and acceptable appearance during an explicit or implicit period of time without requiring unforeseen high costs for maintenance and repair.

Measures necessary to ensure the required service life are chosen according to the environmental conditions and the significance of the structure.

This means that the performance of the structures and structural components in their anticipated working environment should be such that deterioration of material properties or structural components will not lead to an unacceptable probability of failure nor to an unacceptable service performance or appearance.

Service life depends equally on the behaviour of structural and non-structural elements. Both shall be considered during design, construction and use of the structure.

Non-structural elements such as drainage, joints, bearings, installations etc. may require specialist attention other than that of structural engineering. Particular structural components such as anchorages, couplers and deviators for prestressing tendons and their location in the structure may require particular attention.

The avoidance of durability problems throughout the expected life of a structure requires the co-ordinated efforts of all parties involved in all phases of the planning, construction and use of the structure.

The whole process of creating structures and keeping them in satisfactory use and service requires co-operation between the following four parties

- the owner, by defining his present and foreseen future demands and wishes, if any
- the designers (engineers and architects) by preparing design specifications (including proposed quality control schemes) and conditions
- the contractor who should follow these intentions in his construction works; most commonly also subcontractors are involved
- the user, who will normally be responsible for the maintenance of the structure during the period of use.

Any of these four parties may, by their actions or lack of actions, contribute to an unsatisfactory state of durability of the structure and thus cause a reduction of the service life. Also interactions between two parties may cause faults which can have an adverse effect on durability and service

8.1.1. Design strategy

Prior to commencement of design, the owner together with the designer shall determine the required service life of the structure and the required exposure classes.

The design options presented in this chapter can accommodate very short as well as very long design lives of the structures for any of the exposure classes presented in subsection 1.5.2.

National or regional regulations may govern these decisions.

The Model Code clauses on design, execution and maintenance will generally lead to a service life of the structure in excess of 50 years according to section 1.5.

However, some structures will require a substantially longer service life, say 100 years or more, and other structures may need a considerably shorter service life, say less than 25 years.

A structure shall be designed and constructed as good and as robust as necessary in order to satisfy the required service life with a minimum amount of foreseen maintenance.

Accessories such as drainage, joints, bearings, railings, connections, installations etc. usually have a shorter service life than the structure itself, and adequate provisions for maintenance and replacement of such elements should be provided in the design.

The design should consider detailing which increases self-protection and robustness of the structure against aggressive environment.

This includes provisions to ensure satisfactory weathering and ageing of exposed surfaces thus allowing buildings to grow old gracefully without expensive maintenance. An appropriate selection of structural form should be ensured at an early, conceptual stage of the project.

The required service life should be obtained without relying on special protections needing frequent maintenance or redoing.

However, in cases of especially aggressive environments special protective measures may be foreseen, see subsection 8.4.6.

The service life of special protective measures is usually shorter—at times very much shorter—than the intended service life of the structure.

Surface treatments shall be chosen very carefully. The effects cannot always be foreseen in full, and very adverse effects may be experienced.

The Service Life Design Concept is based on a simple two-phase modelling of the technical ageing and deterioration of structures (see Fig. 8.1.1).

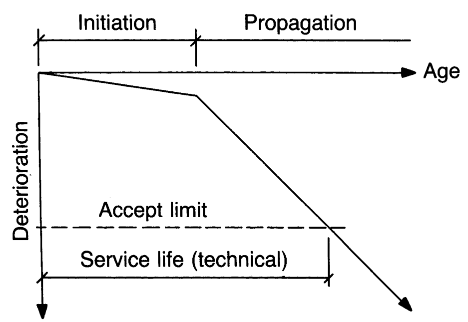


Fig. 8.1.1. Technical service life

During the Initiation Phase no noticeable weakening of the material or of the function of the structure occurs, but some protective barrier is broken down or overcome by the aggressive media of the environment. Examples are carbonation, chloride penetration, sulphate accumulation or leaching of lime.

During the Propagation Phase active deterioration normally proceeds rapidly and in a number of cases at accelerating pace.

The protective measures available may influence favourably either the duration of the initiation phase or the rate of propagation of the active deterioration.

The strategy of the service life design is to select intelligently an appropriate number and types of co-operating measures to ensure the required service life, considering the environment in question.

Robustness in the design is achieved by focusing primarily on extending the initiation period as far as possible towards the required service life, and by ensuring a relatively low rate of propagation if active deterioration should develop within the actual period of use.

The design strategy should consider possible measures to protect the structure against premature deterioration. A set of appropriate measures (one or more) can be combined to ensure that the required service life is obtained with a sufficiently high probability.

This design strategy is considered a 'Multi-Stage Protection Strategy' which leaves the selection of individual protective measures to the designer.

The different measures may act simultaneously in contributing to the protection, or one measure may be substituted by the next, once the former has been overcome, eliminated or surpassed by the aggressive substance.

Protective measures may be established by, *inter alia*

- the selected structural form
- the concrete composition, including special additions or admixtures
- the reinforcement detailing including concrete cover
- a special skin concrete quality, including skin reinforcement
- limiting or avoiding crack development and crack widths, e.g. by prestressing
- additional protective measures such as tanking, membranes or coatings, including coating of reinforcement
- specified inspection and maintenance procedures during in-service operation of the structure, including monitoring procedures
- special active protective measures such as cathodic protection or monitoring by way of sensors.

A different level of reliability is associated with the protective effect of each type of measure. This level of reliability depends much on the quality assurance scheme associated with the establishing and possible maintenance of each protective measure.

A secondary effect is associated with e.g. the selection of epoxy coated reinforcement with individually coated bars, which rules out later use of cathodic protection.

The choice of protective measures shall be carefully considered in relation to the particular aggressive environment encountered. Possible secondary effects shall be evaluated.

An unfavourable secondary effect could be the increased rate of carbonation following water repellant impregnations.

A characteristic example of multi-stage protection is post-tensioned structures in a very aggressive environment (high chloride content) where the different protective stages—or measures—could be

(*a*) tanking, coating or a stainless steel lining on the concrete surface
(*b*) a highly impermeable (low water/cement ratio) concrete with pozzolanic additions
(*c*) reasonably large and crack-free concrete cover
(*d*) protective sheathing or ducts, metallic, polyethylene or similar
(*e*) cementitious grout; for unbonded tendons the grouting could be made with corrosion protective grease.

In this example (*a*), (*b*) + (*c*), (*d*) and (e) act consecutively, whereas (*b*) and (*c*) act simultaneously. Also (*b*) as well as (*e*) (with cementitious grout), containing two protective elements each, are acting simultaneously.

For a non-prestressed structure in a similar environment the protective stages could be

— (*a*) and (*b*) unchanged
— (*c*) not considered (as crack-free concrete cannot be ensured)
— (*d*) epoxy coated reinforcement or stainless steel reinforcement or foreseen cathodic protection.

In both cases, an alternative to (*a*) could be larger, high quality concrete cover with low permeability and with a separate small diameter skin reinforcement made of epoxy coated bars or stainless steel, to ensure crack distribution and avoid longitudinal cracking.

The design shall, wherever possible, ensure adequate access to all parts of the structure, including voids and accessories, to allow for inspection and possible maintenance to be performed throughout the intended service life of the structure.

The design shall take into account the execution and maintenance policy foreseen for the structure. In case of doubts, the design strategy should be modified accordingly.

8.1.2. Execution

The reduced quality of execution and curing shall be thoroughly specified and subsequently controlled and documented during the construction phase. The quality control procedures should follow a preplanned quality assurance scheme adjusted in detailing and technical level to

- the type, shape, complexity and sensitivity of the structure
- the type and aggressivity of the (local) environment
- the experience and competence of the contractor.

The chosen quality assurance system should provide a high probability of the structure being made right in the first place.

All numerical values given are to be considered as minimum measures and should not be violated.

The quality of execution including curing has a dominant influence on the quality of concrete and on the dimensions, such as cover, obtained in the structure.

The intention is that violation of one minimum measure should not be attempted and compensated for by going beyond the minimal value of another measure, such as increased concrete cover to compensate for reduced concrete quality (e.g. higher water/cement ratio).

8.1.3. Use and maintenance strategy

Regular and systematic inspection of the structure and all accessories shall, where possible and relevant, be exerted throughout the intended service life.

A pre-chosen maintenance strategy influence the decisions regarding structural design and layout. The strategy should be decided upon prior to the commencement of the design.

Results of the findings during inspection may determine the subsequent intervals between inspection and their intensity.

In some cases inspection is impossible or very complicated, e.g. for foundations. Robust design and intensified quality control during execution may then be the only available measures.

Accessories usually exhibit a shorter service life than the structure itself and maintenance, repair and replacement must be foreseen in the design and during use.

8.2. DETERIORATION MECHANISMS

The combined transportation of heat, moisture and chemical substances, both within the concrete mass and in exchange with the surroundings (micro-climate), and the parameters controlling these transport mechanisms, constitute the principal elements of durability. Transport mechanisms are treated in detail in subsection 2.1.9.

The presence of water or moisture is the one single, most important factor

In applying these criteria, the designer has an interest to keep in mind the essential deterioration mechanisms considered in this Model Code. These mechanisms are presented in detail in the CEB Design-Guide to Durable Concrete Structures, Bulletin 182.

anical deterioration. The transport of water within the concrete is determined by the pore type, size and distribution. Thus, controlling the nature and distribution of pores becomes an essential task during the initial process of creating concrete structures.

In turn, the type and rate of degradation processes for concrete (physical, chemical and biological) and for reinforcing or prestressing steel (corrosion) determine the resistance and the rigidity of the materials, the sections and the elements making up a structure. Also the surface conditions are determined in this way, and this is reflected in the safety, the serviceability and the appearance of a structure (i.e. determines the performance of the structure).

8.3. ENVIRONMENTAL CONDITIONS

8.3.1. Exposure classes

Environmental conditions mean those chemical and physical actions to which the concrete structure is exposed and which result in effects that are not considered loads or action effects in structural design.

In absence of a more specific study, these environmental conditions may be classified into the exposure classes given in Tables 1.5.1 and 1.5.2 of section 1.5.

8.3.2. Micro-environment

Classification of environmental conditions (Table 1.5.1 in subsection 1.5.2) should be related to micro-environment and not to macro-environment.

The micro-environment to which a building material or component is exposed reflects the macro-environment acting at the location of the structure, modified by the structure itself.

The micro-environment is the environment in the immediate vicinity of the point considered on the surface of the structure or the structural component.

The micro-environment may differ considerably from the original macro-environment.

Concrete absorbs water quicker than it dries out. The water content in the surface layers will thus usually be higher than corresponding to the average relative humidity of the environment. Due to the hygroscopic effects, this tendency is even more pronounced if the concrete contains some chlorides, either accidentally mixed into the concrete (as polluted aggregates or mixing water, or as an accelerator) or penetrated from the outside, i.e. as de-icing salts or from spray of seawater.

In most cases a cyclic wetting and drying (seasonal changes or on a day-by-day basis) will provide sufficient moisture to allow deterioration to develop.

Water will especially accumulate at intersections between exposed horizontal and vertical surfaces where dust and dirt may accumulate and water is maintained, thus keeping the concrete wet locally over an extended period of time.

The different orientations of structural components will experience different amounts of wind, sunshine and driving rain. In the northern hemisphere southern and western orientations usually have higher maximum temperatures, higher temperature variations, stronger effect from UV-light, and also are more often affected by driving rain. This changing microclimate may promote fast carbonation, quicker corrosion once started and rapid deterioration of a surface protective coating. This may reflect on the quality of concrete selected or on the maintenance strategy adopted.

Northern and eastern orientations may exhibit slower drying-out which may increase risk of frost damage and may increase the tendency of vegetation and bacteriological growth on the concrete surface.

The regular washing and drainage of water from the façades is decisive of the long-term appearance of structures.

The appearance of the structure is much affected by the miscolouring caused by dust, dirt and soot depositing on the concrete surfaces, especially the vertical ones.

On high rise buildings driving rain will only affect the top 2–3 storeys. The remaining stories will not be noticeably affected, i.e. a washing of the façade due to rain cannot be relied upon.

In the design, attempts shall be made to minimize the possibilities of such depositing and to profit from the natural clean-washing which occasionally may be provided by driving rain. This requires a carefully planned channelling of water run-off on the surfaces of structures.

Exposed aggregate surfaces with high quality impermeable coarse aggregate have a relatively high degree of self-protection or self-rinsing ability. This is partly due to the low percentage of the surface being exposed mortar with high capillary effect, partly due to such a surface providing a certain visual camouflage of the deposited dirt.

Window sills, balconies etc. protruding from the walls cause shadow effects on the wall section below, thus preventing possible washing of the wall by falling rain. Very unsightly dirt patterns may result.

Some concrete compositions and surface textures are more sensitive to dirt depositing than others. This shall be considered at the initial choice of materials and texture of exposed surfaces.

The different aggressivity of the micro-environments of a structure may be taken care of in the design by applying a zoning strategy. According to this, the structural elements, or the most adversely affected parts of the elements may be considered as belonging to different zones, depending on the aggressivity of the micro-environment foreseen.

A division in two or three zones can be recommended, i.e.

zone 1: slightly susceptible
zone 2: moderately susceptible
zone 3: highly susceptible.

The substance causing the aggressiveness may vary from structure to structure, or even within different parts of the same structure.

Usually the design and execution should provide a high quality structure to all zones, but special provisions can be added for the more susceptible zones. Such provisions could be the application of a surface protection with the more intensified maintenance associated with such provisions. Alternatively a stainless steel lining, epoxy coated reinforcement or easy replacement of the most exposed structural components may be options to choose between.

Accessories shall be maintained regularly at intervals determined individually depending on the type of structure, its use and its environment.

8.4. DURABILITY DESIGN CRITERIA

8.4.1. General

The design strategy of this Model Code vs. durability is dissuasive, i.e. the satisfaction of the basic requirement of durability is secured by means of appropriate anticipatory measures, which ensure (with an acceptable probability) that the behaviour of the structure in the ULS and the SLS will not be finally affected during its service life.

In order to satisfy the requirements of section 8.1 the following criteria should be used.

(*a*) An appropriate structural form should be selected at an early stage of the project, in order to avoid disproportionately sensitive structural arrangements and to secure adequate access to all critical parts of the structure for inspection and maintenance.

The relevant rules are described in subsection 8.4.2.

(*b*) An appropriate quality of concrete in the outer layer ('skin') of the structural elements shall be secured. A dense, well compacted and well cured, strong and low permeability concrete is needed, which should not exhibit map cracking. Besides, an adequate thickness of concrete cover should be provided.

The relevant rules are described in subsection 8.4.3.

Low permeability towards gases, moisture and aggressive agents is needed.

(*c*) Adequate detailing of reinforced and prestressed concrete structural elements should ensure the integrity of critical surfaces or corners and edges in order to avoid any unforeseen concentration of aggressive influences.

The relevant rules are described in subsection 8.4.4.

(*d*) Under specified environmental conditions and/or for small diameter reinforcing bars or prestressing single wires, nominal crack widths should be controlled under specified load conditions to avoid depassivation during the specified design life.

The relevant rules are described in subsection 8.4.5.

(*e*) Under strongly aggressive environmental conditions, protective surface coatings may be needed.

The relevant rules are described in subsection 8.4.6.

(*f*) The design for durability should be based on accepted sets of standards for materials and recommendations for execution, and on a given maintenance policy.

Detailed information on concrete technology is presented in Appendix d.

8.4.2. Selection of structural form

The selected structural form of exposed concrete structures has a decisive influence on the interaction between the structural material and the environment.

An architectural design well thought of from the point of view of required long service life may lead to considerable improvements in the durability and appearance of concrete structures.

All exposed concrete surfaces should be adequately drained. Only preplanned ponding may take place.

Complexity in structural form, as well as in execution and use, will usually increase the sensitivity of the structure to deterioration, shorten service life or require increased efforts in maintenance. Also the ageing characteristics may deteriorate with increased complexity.

In the selection of structural form adequate care should be taken to provide robustness against deleterious liquid or gaseous substances penetrating into the structure.

Most deterioration mechanisms are governed by some substance which in liquid, dissolved or gaseous form penetrates from the surrounding environment through the concrete surface into the concrete. Exceptions are physical and some mechanical damage.

Water may trigger inherent deleterious reactions and may cause leaching of lime, thus causing dissolution of the cementitious effect in the concrete.

The geometry of exposed structural components and the form, type and placing of joints, including construction joints (see subsection 8.4.4), connections, and supports should be chosen such as to minimize the risks of local concentrations of deleterious substances. These concentrations may develop on the surface of the structure as well as within the concrete when these substances enter the concrete by permeation, diffusion, capillary action or similar.

The robustness of an exposed structure or structural component is partly related to the ratio between the exposed surface area and the volume of concrete. The larger this ratio the greater is the risk of some deleterious substance penetrating into the concrete in sufficient quantity to initiate deterioration of the concrete or the reinforcement. For building façades exposed only on one side, the ratio between the exposed surface area and the projection of the façade on a vertical plane is a relative measure of the vulnerability of the structure.

The higher the risk of deterioration (relative risk factor) the more the need for care in selecting the concrete composition, the concrete cover, the correct execution and curing, and an adequate and appropriate maintenance scheme.

Near out-going edges and corners aggressive substances can penetrate into the concrete from more than one side thus leading to local concentrations. If the concrete or the reinforcement is prone to deterioration under the prevailing environment, this will lead to early development of damage at the out-going corners and along such edges, i.e. a so-called corner effect develops.

Near in-going edges and corners stress singularities in loaded structural components occur, and the risk of cracking is increased. This may increase the rate at which aggressive substances, including water, locally penetrate into the concrete.

Cyclic wetting and drying effects will strongly accelerate the rate at which dissolved aggressive substances enter the concrete and concentrate near the surface of evaporation.

In these cases the selection of rounded corners and edges will reduce the concentration effects and thus enhance the durability of the structure. Examples are: round columns or columns with rounded side faces in an aggressive environment are more durable than square or rectangular columns; the same goes for rectangular beams.

The exposed edges should be rounded with a radius of curvature of minimum three times the nominal concrete cover, but depending on the bar diameters. A similar curvature should be found in the transverse reinforcement. The corresponding longitudinal corner reinforcement should also be distributed over the rounded edges.

Drainage of water over concrete should be limited as much as possible, and drainage over joints and seals should be avoided.

Watertight joints and seals cannot be considered fully watertight throughout the service life of the structure. Consequences of temporary leaks, e.g. at time up to maintenance or replacement, should be foreseen in the secondary protection or drainage of underlying elements.

This may lead to slopes draining the upper surface of supporting beams or columns. Also special water protection or drainage may be applied on these areas. Where de-icing salts are used, e.g. on bridges, parking decks or balconies, this secondary protection is particularly important due to the risk of chloride corrosion following a leak.

Care should be taken in the detailing of façades of buildings and structures in order to allow easy drainage of water and to facilitate clean-washing.

Roofs with large eaves provide valuable protection of the façades against wetting. Bands of balconies and galleries may have a similar effect.

Surface areas subjected to wetting, splashing or water accumulation should be kept as small as possible.

Retaining walls and bridge piers close to traffic roads may profit from having a larger distance to the road than the required minimum. This will reduce the splash water and fog spray caused by the traffic. This is especially

valuable in the case of frequent use of de-icing salts on the road. The possible increase in cost for the structure may well turn to an economic advantage in the long run.

Alternatively special protection may be applied, see subsection 8.4.6. If the special provision shall protect against corrosion risk due to penetrating chlorides, epoxy coated reinforcement or cathodic protection (sacrificial anode or imposed current systems) are options to be considered.

Most special provisions seem to have a service life of their own which is shorter than the stipulated service life of the structure they are to protect. Consequently such provisions may only be relied upon to ensure the required service life if regular inspection and maintenance is carried out as an integral part of the protection.

Placing and special protection of anchorages for prestressing tendons should be selected considering the local exposure.

Horizontal faces where ponding may take place, and vertical faces on which water may drain are especially risky zones for placing anchorages for prestressing tendons. Even if the anchorage zone is protected by seals or watertight joints etc. it should be foreseen that seals and joints may leak and shed water over the anchorage zone. This should be foreseen in the selection of anchorage protection system.

If possible, internal holes and voids in the structure should be drained and ventilated.

Unintentional voids shall be avoided. This is especially important for post-tensioned concrete structures where lack of grout or insufficient grouting of the ducts may leave ungrouted portions of the tendons.

Water may accumulate in any void present within an exposed structure. Leaks, cracks and condensation may contribute to this accumulation. This water accumulation may cause corrosion on exposed reinforcement. Prestressed reinforcement in grouted ducts especially prone to such unwarned corrosion in the case of unintentionally ungrouted parts of the ducts.

Good and complete grouting of prestressing ducts should be ensured.

Water may also fill up the void or the ungrouted portion of ducts. When later subjected to freezing, severe spalling or bursting of the element may occur.

Structural conditions likely to lead to large local cracks should be avoided. Risks of such cracking should be handled by an appropriate detailing of the reinforcement.

Abrupt deviation of forces in a structure and abrupt changes in section causes stress concentrations likely to cause single large cracks. Concentrated forces due to anchoring of prestressing tendons or due to reactions from supports create large local splitting forces which may cause large cracks. Restraining forces due to differential settlements, shrinkage and temperature effects may also cause large cracks.

8.4.3. Concrete materials, cover thickness and prestressing tendons

Whenever a specially conceived and approved durability assurance scheme is not available, the following rules are deemed to provide a highly impermeable concrete in accordance with subsection 8.1.1.

(a) Concrete composition
The requirements stated in d.6 in Appendix d shall be followed. The exposure classes of subsection 1.5.2 determine the corresponding requirements to be followed.

Appendix d gives the elements among which the design shall choose a combination able to satisfy the specific requirements of the individual structure.

Usually the choice may comprise the following elements

- types and strength class of cement, see d.6.3.3.1
- additions, see d.6.3.3.2 and d.6.5.3
- aggregates, see d.6.3.3.3 and d.6.5.1
- cement contents and water/cement ratio see d.6.3.4, Table d.2 and d.6.5.2.

Concretes with special properties are dealt with in d.6.6.

Special care shall be taken to ensure a high quality impermeable concrete in the outer layer—or 'skin'—of the structure.

Casting and curing conditions have a decisive influence on the permeability of this 'skin'-concrete.

The requirements differ depending on the type of structure, it being

- plain concrete
- reinforced concrete
- prestressed concrete.

The sensitivity to deterioration mechanisms may increase going from plain concrete to prestressed concrete.

Admixtures containing harmful chloride contents should not be applied to reinforced or prestressed concrete structures (see Appendix d).

For indoor structural parts and for structures in prevailing dry environments the relative humidity may be kept permanently at a level where chloride induced corrosion may occur. However, short-term, temporary or local admittance of moisture or water to the concrete may suffice to initiate corrosion and later maintain a corrosion process. Such cases may occur during construction, in kitchen and bathroom environments, at leaks in pipes and after change of use of the structure. In warm dry environments this may occur during occasional rains, due to irrigation, near water outlets, from air-conditioners, and during the fall of dew.

(b) Spacers

Spacer material shall be selected with due regard to the aggressivity of the environment.

In exposure class 3–5 spacer material should preferably have good adhesion to the concrete.

Requirements to concrete quality in the outer layer—or 'skin'—of the structure shall also be satisfied for concrete spacers.

(c) Minimum cover

The minimum distance between any concrete surface and the nearest reinforcement bar, or the nearest prestressing tendon or the sheathing for such tendons, shall be obtained from Table 8.4.1.

The values are absolute minimum values with no downward tolerances allowed and no upward tolerance being specified.

The nominal values, c_{nom}, are equal to the minimum values plus tolerance according to the rule

$$c_{nom} = c_{\min} + \text{tolerance}$$

Tolerance should be taken as 10 mm unless in the individual case it can be demonstrated that a lower value is obtainable. The tolerance should not be less than 5 mm.

Spacers shall be designed according to c_{nom}.

The spacing of spacers shall be adequate to ensure the required cover.

When controlling concrete cover after placing and hardening of concrete the measured values may not be less than $c_{\min}$.

Table 8.4.1. Minimum cover, $c_{\min}$

Exposure class	$c_{\min}$ (mm)
1	10
2	25
3, 4	40
5	*

*Depends on the individual type of environment encountered.

The values in Table 8.4.1 refer only to corrosion protection of reinforced and prestressed concrete structures.

Other reasons (see chapter 9) may warrant larger covers such as

- ensuring bond strength
- ensuring fire protection
- use of larger aggregate sizes.

(d) Protection of bonded prestressing tendons

In accordance with subsection 8.1.1 the corrosion protection of prestressing tendons may consist of several successive protection measures such as

- quality and diameter of prestressing steel
- grouting of ducts or sheathing
- type of sheathing

- quality of concrete cover
- thickness of concrete cover
- special surface protection
- sealing of anchorages.

Each step may be individually chosen and designed to obtain optimal combined protection for a given environment, in order to obtain the required service life with an acceptable level of reliability.

Prestressing steel and sheathings shall be adequately protected against environmental influences and mechanical damage during transport and storage and during the period between assembly and grouting, according to subsections 11.5.1–11.5.5.

(e) Protection of unbonded tendons

In accordance with subsection 8.1.1 the corrosion protection of unbonded tendons may consist of several successive protection measures such as

- quality and dimension of prestressing steel
- type of sheath filling material such as grease, wax or other suitable agent
- type of sheathing, depending on whether tendons are internal or external
- for internal tendons: quality and thickness of concrete cover
- special surface protection
- sealing of anchorages.

Each step may be individually chosen and designed to obtain optimal combined protection in a given environment in order to obtain the required service life with an acceptable level of reliability.

The prestressing steel shall be effectively protected over its entire length including the anchorages and any coupler present considering the chosen exposure class and the intended service life of the tendons.

When various types of materials are combined it shall be verified that they do not contain any substance harmful to the prestressing steel and that materials which come into contact with each other are compatible.

Special attention is to be paid to the corrosion protection of the anchorage. The continuous protection of the prestressing steel shall be ensured under due consideration of the relative movement between prestressing steel and anchor head during the stressing operations.

A distinction should be made between internal and external unbonded tendons.

For internal tendons the properly specified and executed concrete cover provides the primary corrosion protection. Supplementary corrosion protection barriers are provided by the sheathing and the long-term cement grout, grease, wax or other suitable agent used as sheath filling material.

For external tendons the sheathing provides the primary corrosion protection. A supplementary corrosion protection barrier is provided by the long-term cement grout, grease, wax or other suitable agent used as sheath filling material.

The principles for selecting protective measures are similar for monostrands and for multiple tendons.

The sheathing shall be completely waterproof and continuous for the full length of the tendon, and shall prevent the intrusion of cement paste during concreting.

The sheathing material should be of a type which will not react with cement, grease or steel. It should be durable and resistant to damage and abrasion and it should remain stable and flexible during handling, storing on site and in service for the range of temperatures likely to be experienced.

The sheathing should maintain sufficient strength and flexibility to bridge any fine cracks which may appear in the concrete.

The shape of prestressing strands should not be reproduced on the outside of the sheath.

The sheathing material shall be resistant to ageing by exposure to UV-light.

Sheathing may not contain PVC.

PVC can produce chlorides and may increase the corrosion risks.

The anti-corrosion grease, wax or similar void filling agent shall be impervious to moisture, non-absorbing and not capable under static conditions of forming an emulsion with water.

The force required to move the prestressing steel in the finished protected sheathing in its final location in the structure shall be limited to allow subsequent stressing to take place.

The force required to move the prestressing steel in a 1 m long sample at room temperature should not exceed 75 N.

Corrosion protection filler materials injected into the sheathings or pipes (multiple tendons) shall satisfy the following requirements

- easy to inject into the pipes in more or less fluid form

A higher temperature of the filler should neither adversely affect the duct nor the prestressing steel.

- able to restabilize in a way that no leakages occur under the prevailing conditions
- have a known thermal expansion coefficient

It shall be decided whether expansion chambers are needed to cope with higher ambient temperatures.

- be of good tightness and provided with vents in sufficient number and at locations to prevent air pockets and voids.

The tightness of the tendon envelope, especially at the anchorages, and the ambient temperature shall be taken into account.

Unbonded tendons and sheathings shall be adequately protected against environmental and mechanical damage during transport and storage, according to subsections 11.5.6 and 11.5.7.

8.4.4. Detailing

Structures should be designed taking into account the combined effects of the relevant exposure class of subsection 1.5.2, the detailing of the structural form including joints, connections and supports as stated in subsection 8.4.2 and the detailing of the reinforcement layout.

Congestion of reinforcement may lead to difficult casting conditions where the concrete is segregated by being sieved through the reinforcement, thus causing bad compaction and honeycombing.

The reinforcement, non-prestressed as well as prestressed, should to the extent possible be sufficiently distributed within the concrete zone foreseen for this purpose to ensure a good and reliable casting and compaction of the concrete, especially in the outer concrete layer constituting the skin of the concrete structure.

Bundled reinforcement should comply with the requirements for cover, spacing and bundling as stated in subsection 9.1.5.

Where possible the reinforcement layout should constitute a closed three-dimensional reinforcement cage. When using links or stirrups, these should have a geometric form which complies with the three-dimensional cage action.

The three-dimensional closed reinforcement cage action provides a confinement of the concrete, and can increase the structural reliability in the case of any deleterious expansive reactions within the concrete causing delamination and splitting of the concrete. This increases the overall robustness of the structure.

In the case of slabs which may need no shear reinforcement a minimum number of uniformly distributed links may be provided to ensure a sufficient three-dimensional confinement of the concrete in case of any splitting or delamination of the bulk concrete.

In moist environments the cast-in steel area determines the rate of corrosion of the exposed part of the steel.

Structural components exposed permanently or temporarily to moist environments, such as steel inserts, temporary hooks, fasteners etc. cast into the concrete, shall not be in electric contact with any cast-in reinforcements.

The corrosion rate of the exposed part of the steel may be further reduced by coating the steel item, either the cast-in part only or, if possible, the total steel item.

These requirements are also valid for joints and bearings.

Alternatively stainless steel may be applied, in which case electric contact to the reinforcement is admitted.

Construction joints should be located in parts of the structure where the exposure to water and dissolved aggressive substances is minimal. Any water expected to run on the surface should preferably run parallel to the joint.

In exposed structural parts construction joints should be selected with due regard to the

— type of exposure and aggressiveness of the environment at the joint

If possible, construction joints should be located in zones with no or minimal tensile stresses in the concrete and with minimal stress variations. Stress reversals in joints should be avoided, if possible.

— the stress level and stress variations foreseen at the location of the joint

In accordance with the basic requirements of subsection 1.5.1, and as stated in section 8.1, acceptable appearance is one of the design objectives. Construction joints are clearly visible and, where relevant, care should be taken in selecting their location and in selecting the formwork determining the visual transition in the surface texture.

— the impact of the joint on the visual appearance of the structure.

8.4.5. Nominal crack width limitations

Basically different strategies need to be applied for prestressed and ordinary reinforced concrete elements.

Due to the different sensitivity of prestressing steel and ordinary reinforcing steel to corrosion induced failures, different corrosion protection needs to be applied.

Corrosion risk and corrosion rate in the region of cracks depend predominantly on the impermeability and the thickness of concrete cover. Therefore, the requirements of subsection 8.4.3 are of special importance to ensure sufficient corrosion protection of the reinforcement in cracked regions.

This has been shown by laboratory tests and practical experience.

For structural elements exposed to environments according to exposure class 2 to 5 (see Table 1.5.1 in section 1.5) crack width limitation for prestressed elements shall ensure that the prestressing steel will not be depassivated during the anticipated service life. The limitations given in Table 7.4.1 (section 7.4) normally are satisfactory.

Prestressing steel may fail due to stress corrosion cracking (SCC) or hydrogen embrittlement (HE) in a very brittle manner and without warning even at rather little degrees of corrosion. The design strategy therefore, shall be to avoid any depassivation of the prestressing steel surface if the environmental conditions may lead to corrosion.

Zinc coated or galvanized sheathings may not be used for grouted ducts as hydrogen develops and greatly increases the risk of HE.

For structural elements or sections being affected by imposed deformations, e.g. temperature gradients, not covered by the frequent load combination and leading to cracking across the prestressing steel, additional measures compared to Table 7.4.1 are necessary. Additional measures could be

- impermeable ducts or coating
- compression within the whole section.

Normally the requirement 'decompression' is sufficient to ensure that cracks due to e.g. temperature gradients within the sections not covered by direct calculations, will be limited to the surface areas of the concrete, and will not cross the prestressing steel. However, this may not be ensured in more complicated cross-sections like box girders. Temperature gradients may arise e.g. from sunlight.

If the prestressing steel is located in deeper sections cracking of the outer concrete layer is acceptable if the crack depth is limited to an extent that ensures an uncracked cover over the prestressing steel according to the defined nominal values given in Table 8.4.1.

Depassivation, primarily due to carbonation, cannot be totally avoided in the region of cracks crossing ordinary reinforcement. Crack width is of minor importance with respect to corrosion rate in the range to be expected, if the design is in accordance with the Model Code.

Test results and practical experience show that crack width does influence the time to depassivation, however a limitation of crack width to avoid depassivation during the whole service life is impossible for ordinary reinforced elements. After depassivation crack width practically does not influence the corrosion rate. Therefore, a differentiation of permissible crack widths depending on environmental classes is not considered necessary for ordinary reinforced concrete elements.

In the case of very severe chloride and unfavourable structural conditions (e.g. cracks due to tension in horizontally oriented elements with chloride attack on the top side) special protective measures like coating of concrete or coating of reinforcement should be taken. Limitation of crack widths to lower values than $w = 0.3$ mm cannot prevent corrosion damage under those unfavourable conditions.

Very severe chloride attack occurs in, for example, unprotected park decks with restraint induced cracking due to shrinkage and/or temperature. Chloride containing water penetrates through those cracks even at very low crack widths. Temperature changes may avoid healing of even small cracks.

8.4.6. Special protective measures

The required service life of concrete structures should primarily be obtained (see subsection 8.1.1)

- through a robust structural design taking the specific environmental aggressivity into account when selecting the structural form
- by selecting an appropriate concrete mix
- by ensuring an adequate compaction and curing of the concrete
- by selecting an inspection and maintenance strategy which will reveal the condition of the structure in use in due time for preventive maintenance or minor repairs to be performed before costly repairs are needed.

In case of especially aggressive environments where the normal provisions to ensure the required service life cannot suffice, and in cases where insufficient durability has resulted in damage to an existing structure, special protective measures may be applied to obtain the required service life.

The special protective measures are of the following types.

- Provide smooth surfaces and minimize the area exposed to the environmental aggressivity.
- Provide structural protection such as
  - — roof, eaves or similar to protect concrete surfaces against rain
  - — surface protection in the form of a water repellant impregnation, a thin or thick film coating, tanking (of e.g. foundations), membrane and lining

Surface coatings may be selected barriers against penetrating substances such as

- water (H_2O)
- water vapour
- carbon dioxide (CO_2)
- chloride ions (Cl^-)
- oxygen (O_2).

The consequences of selecting such barriers should be carefully evaluated, especially the risks of possible accumulation of moisture in concrete with a water impermeable membrane. As an example alkali-aggregate reactions may develop even in old structures following surface treatment, if the concrete contains alkali-reactive aggregates.

Chlorides enter into the concrete dissolved in water; this is why water repellant or water impermeable membranes should be used to prevent the ingress of Cl^-.

Theoretically corrosion can be stopped by preventing access of O_2 to the

reinforcement. However, in general this is an unreliable procedure. In practice corrosion inhibition can be obtained only in fully submerged structures. The ageing of surface coatings and their ability to be re-done at regular intervals should be carefully evaluated prior to selecting products.

The colour stability of coatings may vary, and should be considered prior to selecting products.

A moisture membrane may be protected by a separate overlay, e.g. as on traffic loaded elements subjected to de-icing salts.

Two different strategies may be followed when selecting increased concrete cover with a special skin reinforcement.

(*a*) The cover on the skin reinforcement is considered a sacrificial cover which may spall, if this is acceptable, some time in the future if the skin reinforcement corrodes. In the latter case the skin reinforcement acts as a sacrificial anode protecting the main reinforcement provided they are connected electrically. The outer part of the cover is not taken into account in the load carrying capacity. Stiffness and restraining forces are calculated with and without this extra cover.

(*b*) The skin reinforcement is specially protected, e.g.

(i) *By polymeric coating*. It should be ensured, that there is no electric contact between the coated skin reinforcement and the uncoated main reinforcement.
By maintaining uncoated main reinforcement (coating of single bars) a future installation of cathodic protection is a valuable option, should this prove necessary some time in the future (Multi-Stage Protection Strategy, see subsection 8.1.1).

(ii) *By selecting specific stainless steel.* There is no restriction in electric connections to the main reinforcement.

— increased concrete cover; provide special skin reinforcement if $c_{nom} \geqslant 70$ mm

— reduce environmental aggressivity by, for example, surface insulation, thus controlling heat and moisture conditions in the concrete (housing).

- Provide special protection of the reinforcement, such as

— placing prestressed reinforcement in sheathings (metallic or plastic) with special corrosion protective grout or void filler

— coating of reinforcement

Polymeric coating of reinforcement may provide a long-term reliable barrier against corrosion of reinforcement due to either carbonation or chloride ingress, provided the coating

- has sufficient thickness
- provides a total coverage, allowing only minimum amounts of pinholes and holidays (see special standards)
- has a high and lasting bond to the reinforcement
- is undamaged in bent bars
- is patch repaired sufficiently at cut ends.

and provided coated bars are not mixed with uncoated bars in the same structural component where there is a risk of electric contact between the two types of bars.

Coated bars have different bond and anchorage characteristics which should be taken into account in the design and detailing of the reinforcement.

Galvanizing may prolong the service life of cast-in reinforcement if the concrete carbonates, but galvanizing is not considered to provide a long-term reliable corrosion protection of reinforcement in chloride contaminated concrete.

— cathodic protection.

Cathodic protection of reinforcement may provide a reliable protection against corrosion even in cases where very high chloride concentrations may occur. Such protection may also be achieved for reinforcement where corrosion has started.

Cathodic protection of immersed or buried parts of a structure can be achieved by traditional local anodes placed in water or in moist soil and the protection based on either sacrificial anodes or on an impressed current system.

For structural parts in the air, anodes should be placed on the concrete surface either distributed or localized, and the protection based on an impressed current system. Several surface mounted anodes need a conductive overlay to ensure the current distribution to the reinforcement. The dead load of such overlays should be considered in the design.

Cathodic protection of prestressed structures is considered possible, but due to the increased risk of hydrogen embrittlement of the prestressing steel, care should be taken in the design and especially in the monitoring of the system to avoid overprotection and hydrogen development at the reinforcement.

Cathodic protection tends to increase the concentration of alkalis near the reinforcement. This may increase the risk of alkali-aggregate reactions if the concentrate contains alkali-reactive aggregates.

For new structures anodes may be placed in the formwork and cast integral with the structure. Any short circuits with the reinforcement will be detrimental to the system.

Initial electric continuity of the reinforcement will greatly facilitate the installation of cathodic protection some time in the future, should this prove necessary.

For structures in chloride containing environments (exposure class 3 and 4) all reinforcement should be electrically continuous for the whole structure or individually for selected structural components

— select non-corroding reinforcement (specific stainless steel).

- Provide special monitoring systems (e.g. a warning system) to follow the condition of the structure.
- Provide intensified inspection and maintenance routines to cope with early deterioration.

8.4.7. Prerequisites related to execution and maintenance

The probability of achieving robustness and good quality concrete in the most exposed parts of a structure, such as in the outer concrete skin, is increased considerably by selecting structural form and detailing which facilitates the execution and curing process. This includes

- adequate dimensions to ensure easy casting and compaction of concrete
- formwork which, where relevant, provides smooth and pleasant surface texture
- detailing of reinforcement to ensure high quality, well compacted concrete in the concrete cover, free of any honeycombing, see subsection 8.4.4
- predetermined positioning of construction joints to match the exposure class and the foreseen structural performance.

Structures should be designed and detailed, with due regard to the relevant exposure classes, and with due account of the execution procedures foreseen, such as to

- facilitate execution
- be adequately inspectable
- be maintainable.

The design should be based upon an explicitly stated maintenance strategy.

A quality assurance level for execution and maintenance should be determined prior to finalizing a design.

In general the adequacy and workability of the concrete mix for a specific job should be verified by performing trial castings under conditions which simulate actual conditions during construction. Full verification testing should be made on test specimens from trial castings in accordance with the concrete specifications for the work.

The execution process should ensure a concrete mix which limits bleeding and should ensure adequate moisture and temperature curing of the concrete to avoid plastic shrinkage, cracking and thermal cracking (see chapter 2 and Appendix d).

During design and execution the more vulnerable parts of the structure which may be expected to need intensified inspection and maintenance should be identified.

Such vulnerable parts may typically be associated with

- joints
- bearings
- drainage systems
- miscellaneous installations
- areas where water and dirt accumulate.

Regular and systematic inspection and maintenance routines are integrated parts of ensuring the required service life of structures. Inspectors should determine the current condition of the structure and its components, and can in general not rely solely on visual inspections.

Visual inspections can only reveal ongoing active deterioration such as

- abrasion and erosion
- cracking and spalling
- disintegration of concrete surfaces due to freeze-thaw attack, sulphate attack, alkali aggregate attack, acid attack etc.
- rust staining and other miscolouring.

In order to minimize future maintenance and repair costs the inspection routines should, to the extent possible, reveal approaching deterioriation or lack of adequate performance in due time for preventive maintenance to be applied, i.e. reveal the transition point between initiation and propagation as presented in subsection 8.1.1, Fig. 8.1.1.

Inspection should include selective sampling of information regarding, where relevant

- carbonation depth relative to prevailing concrete covers
- chloride profiles relative to prevailing concrete covers
- potential mapping with excavations ('windows') to calibrate measurements
- coring including thin section microscopy and petrographic analysis to determine concrete quality such as compaction, curing, microcracking, water–cement ratio, air content, reactivity of aggregates
- surface tapping to reveal surface delaminations.

Such testing selected from the above list and performed prior to any deterioration visible with the naked eye may reveal oncoming deterioration in due time for preventive maintenance to be performed.

It is current experience that in the majority of cases the costs for preventive maintenance may be substantially lower than the corresponding repair once visible damage is developing.

Inspections and maintenance procedures should take due account of the relevant deterioration mechanisms as revealed by the testing.

9. DETAILING

Whereas the amount of steel reinforcement and prestressed tendons is determined by applying the models included in the previous chapters, this chapter contains information related to limit values, location and arrangement of steel elements, dictated by considerations other than those accounted for by means of calculation.

Such other considerations may be

- conditions of validity of simplified calculation procedures
- minimum ductility conditions
- qualitative reliability requirements
- functional requirements
- practical durability measures
- friendliness of execution, etc.

A complete design should always account for these considerations, be it by means of sound empirical rules, whenever respective more specific models are not used.

9.1. ANCHORAGES, SPLICES, ARRANGEMENT

The rules in subsections 9.1.1 to 9.1.5 apply to reinforcing steel. Subsections 9.1.6 and 9.1.7 apply to prestressing steel. To ensure that bond forces are safely transmitted and to prevent spalling of the concrete, the minimum cover of any bar, stirrup, tendon or sheathing of diameter ϕ should be at least equal to ϕ.

Detailed guidance is given in the CEB-Application Manual on 'Concrete Reinforcement Technology' (Bulletin 140) and CEB Bulletin 164 'Industrialization of Reinforcement'.

Subsections 9.1.1 to 9.1.3 are applicable in particular to bars with diameter $\phi \leqslant 32\,\text{mm}$.

Supplementary rules are given

- for high-bond bars of $\phi > 32\,\text{mm}$, in subsection 9.1.4
- for bundled bars in subsection 9.1.5.

9.1.1. Anchorages

9.1.1.1. General

The normal methods of anchorage are

- straight anchorages (Fig. 9.1.1(a))
- curved anchorages:
  - — hooks (at 150° to 180°) (Fig. 9.1.1(b))
  - — bends (at 90° to 150°) (Fig. 9.1.1(c))
  - — loops (Fig. 9.1.1(d))
- anchorages with at least one welded transverse bar within the design anchorage length (Fig. 9.1.1(e))
- anchorage by mechanical devices.

Straight anchorages or anchorages with bends are not allowed for plain bars in tension.

Local transverse reinforcement should be provided if bends or hooks are used for compression reinforcement (see clause 9.1.2.1).

For the shear strength of welds, see clause 2.2.5.1b.

For welded meshes made of plain or indented wires, see subsections 6.9.8 and 6.9.9.

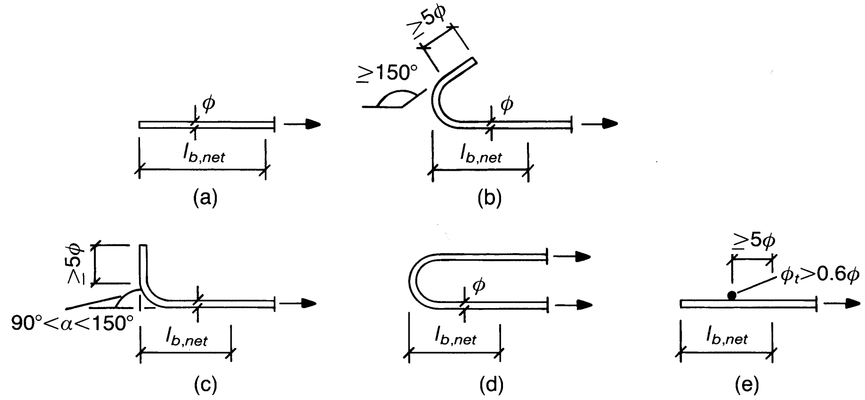


Fig. 9.1.1. Recommended detailing of anchorages

The purpose of transverse reinforcement is to avoid

- longitudinal cracking under the effect of transverse tensile stresses originating in the anchorage zones
- bursting of the concrete due to pressure exerted at the end section of a compression bar (see clause 9.1.2.1).

Normally the minimum shear reinforcement confining the anchored bars (see clause 9.2.2.2) or the minimum transverse reinforcement in slabs (see clause 9.2.1.1), in walls (see clauses 9.2.4.2 and 9.2.4.3) and in columns (see clause 9.2.3.2) fulfils the requirement for minimum transverse reinforcement.

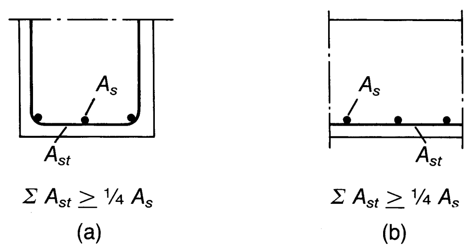
For bars in compression, the transverse reinforcement should surround the bars, being concentrated at the end of the anchorage and extend beyond it to a distance of at least 4ϕ.

Special care is necessary where the bars end near a concrete surface (see Fig. 9.1.3) or are provided with bends or hooks if they are subjected to high compression forces. A minimum cover of 2ϕ is normally required.

Transverse reinforcement should be provided

- for anchorages in tension, if there is no compression transverse to the plane of splitting (e.g. due to a support reaction)
- for anchorages in compression, in all cases.

The minimum area of transverse reinforcement (one leg) is 25% of the area of one anchored bar (see Fig. 9.1.2).



ΣA_{st} is the area of the transverse reinforcement along the anchorage length

Fig. 9.1.2. Minimum transverse reinforcement: (a) beams; (b) slabs

9.1.1.2. Permissible radii of bends

Bars

The diameter of the mandrel used should satisfy the following conditions.

(*a*) The steel bar shall not fail or crack during its bending around the mandrel.

Thus, the minimum mandrel diameter depends on the plastic uniform elongation of the steel under flexural conditions, taking into account rate effects and temperature effects.

Besides, the presence and form of the ribs may also influence the minimum diameter of the mandrel. Because of the high sensitivity of the bars with regard to the parameters mentioned hereover, the relevant indications of the approval documents will apply.

(*b*) Crushing or splitting of concrete, due to the pressure occurring inside the bend, should be avoided under the ULS conditions.

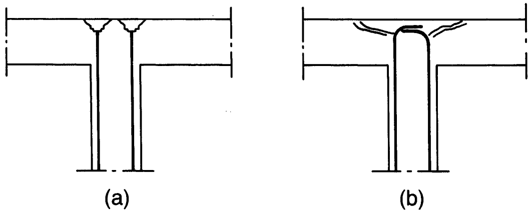


Fig. 9.1.3. Possible damage caused by compression bars ending near concrete surface

In general for ambient temperature, the minimum mandrel diameter should not be less than the diameter required to satisfy the bend–rebend test for the reinforcement.

In the absence of transverse reinforcement, the following simplified model may be used, where the local compression capacity (section 3.3) is reduced by 30% to account for concrete skin deficiencies

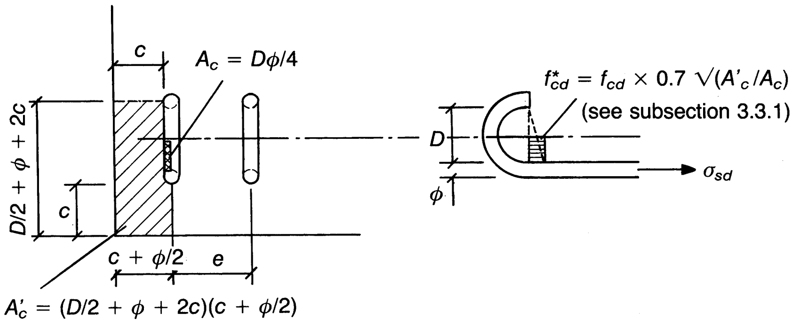


Fig. 9.1.4. Simplified model

The relevant minimum mandrel diameter depends on concrete tensile and steel strengths, on the thickness of concrete cover (or the distance of consecutive bars) perpendicular to the plane of curvature, as well as on the anchorage scheme (hairpin, hook, stirrup angle). Besides, the design value of the steel stress σ_{sd} to be anchored by means of the bent end of the bar has to be taken into account. Last but not least, transverse reinforcement reduces the allowable mandrel diameter (increasing the local compressive strength of concrete).

$$\tfrac{1}{2}D\phi f_{cd}^{*} = (\pi\phi^2/4)\sigma_{sd}$$

$$D/\phi \approx \delta/\sqrt{(1 + 2c/\phi)}(\sigma_{sd}/f_{cd})$$

where $\delta \approx 1.6$. This value (reflecting the f_{cc}/f_{ct} ratio, see subsection 3.3.1, left hand side) should be taken higher for higher concrete strengths.

In the expression above, c = min (cover, e/2), and f_{cd} takes one of the two values defined in clause 6.2.2.2 (generally f_{cd1}).

Similar approximate expressions for other anchorage schemes may be found.

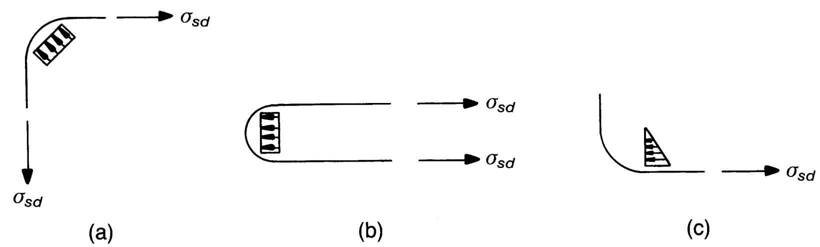


Fig. 9.1.5. δ-values for various anchorage schemes: (a) $\delta \approx 1.6$; (b) $\delta \approx 1.8$; (c) $\delta \approx 2.6$

Vertical forces and moments needed for equilibrium are not represented on the sketches.

Because of the large number of parameters influencing the diameter of the bend, which may lead to concrete crushing, conservative values based on long experience are suggested by National Codes, accounting for specific conditions.

Indicatively, the following minimum values of D/ϕ may be used for 90° bends if 60% of the yield stength of a deformed bar is anchored by the bend. In a conservative way the beneficial role of high strength concrete was not considered in the following table.

Table 9.1.1. Values of D/ϕ for 90°-bends anchoring 60% of the yield strength of a deformed bar—S400; concrete C20

c/ϕ	D/ϕ
$c/\phi > 7$	10
$3 < c/\phi < 7$	15
$c/\phi = 1$	25

c = min (cover thickness, half the free distance between consecutive bars)

If $f_{yk} \neq 400$ MPa the above values of D/ϕ should be multiplied by $f_{yk}/400$.

However, smaller D/ϕ-values may be used if transverse reinforcement is present.

The minimum diameters given in Table 9.1.1 can be used for welded reinforcement bend after welding, provided the distance between the point of welding and the beginning of the curvature is not less than 4ϕ. This distance may be reduced or the welding point may fall in the curved part of the reinforcement if, for structures subjected to predominantly static loading, the mandrel diameter is at least 20ϕ.

For the transmission of the concentrated anchorage force to the concrete, see sections 3.3 and 3.5.

For permissible curvatures, see clause 9.1.1.2.

Welded mesh

The diameter of mandrels of welded mesh bends should respect the two requirements mentioned above, taking into account the additional force transfer capacity due to welded transverse bars.

These rules do not cover cases under predominant dynamic loading.

9.1.1.3. Mechanical anchorage devices

The effectiveness of mechanical anchorages should be demonstrated by suitable tests. The slip, recorded during the test, between the bar and the concrete at the loaded end, shall not exceed

- 0.1 mm under 70% of the ultimate force
- 0.5 mm under 95% of the ultimate force.

The design value of the anchorage resistance should not be larger than

- 50% of the ultimate force of the anchorage, when fatigue loads are negligible
- 70% of the experimental fatigue strength for 10^7 load cycles.

9.1.1.4. Anchorages of stirrups and shear assemblies

The anchorage of stirrups, shear assemblies and confining reinforcement is normally obtained by means of hooks, bends, or welded transverse bars.

The type of anchorage used should not induce splitting or spalling of the concrete cover.

Anchorages by hooks (150° to 180°) are required for plain bars.

Anchorages by bends (90° to 150°) are only allowed for high-bond bars. The anchorage is considered to be fully efficient

- if the curved section of the bar is extended by a straight length of at least
  - 5ϕ or 50 mm following an arc of 150°, or more (Fig. 9.1.6(a))
  - 10ϕ or 70 mm following an arc of 90° (Fig. 9.1.6(b))
- or if on the length of the anchorage there are
  - either two welded transverse bars of diameter not less than 0.7 times the diameter of the stirrup (Fig. 9.1.6(c))
  - or a single welded transverse bar, of diameter not less than 1.4 times the diameter of the stirrup (Fig. 9.1.6(d)).

Shear resistance of welds shall be equal to at least $0.3f_{sy}A_s$ of the anchored bar, when tested naked and $0.5f_{sy}A_s$ of the biggest bar, when tested in concrete.

For arrangement of shear reinforcement, see clause 9.2.2.2.

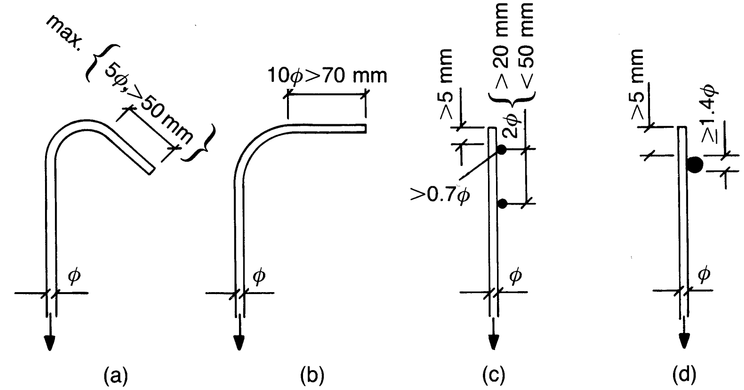


Fig. 9.1.6. Anchorages of links, stirrups and shear assemblies

9.1.2. Splices

9.1.2.1. General requirements for bars

Confinement should be used if bends or hooks are used for compression reinforcement (see clause 9.1.2.2.3b and subsection 6.9.5).

Forces may be transmitted from one bar to another

Splicing of all reinforcing bars of an element at a single cross-section should preferably be avoided (see, however, clause 9.1.2.2.2).

- by lapping the bars, with or without hooks, bends or loops
- by welding
- by mechanical devices assuring load transfer in tension–compression or in compression only.

9.1.2.2. Splices by overlapping

9.1.2.2.1. General requirements

The splices in a section should, if possible, be arranged symmetrically and parallel to the outer face of the element.

Lapped splices should preferably not be placed in zones where the reinforcement is utilized to its design strength.

Force transfer between spliced bars should be safely secured. To this end

- If the transverse distance between two spliced bars does not exceed 4ϕ, the anchorage length should be taken from eq. (6.9–7) (see Fig. 9.1.7(b)).
- If the transverse distance between spliced bars is larger than 4ϕ, the following measures should be taken.
  - — The anchorage length should be increased by a distance equal to the spacing of the bars.
  - — The transverse reinforcement needed should be calculated by a strut and tie model according to Fig. 9.1.7(a); however, this reinforcement should not be lower than foreseen in clause 9.1.2.2.3.

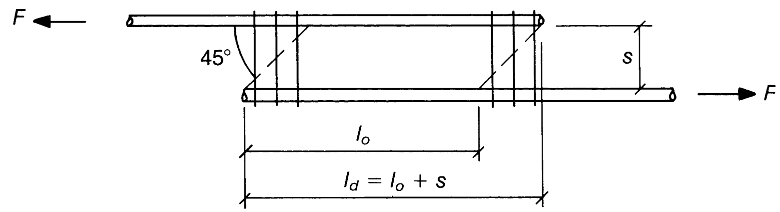


Fig. 9.1.7(a). Splice when $s > 4\phi$

- In principle, lapped splices should be staggered; exceptions are allowed under the conditions of clause 9.1.2.2.2.

  If two adjacent laps are arranged in different sections, the distance separating the ends of bars of adjacent laps should not be smaller than (see Fig. 9.1.7(b))
  - — in the transverse direction 2ϕ and not less than 20 mm (clear distance)
  - — in the longitudinal direction $0.3l_o$ (l_o from eq. (6.9–7)).

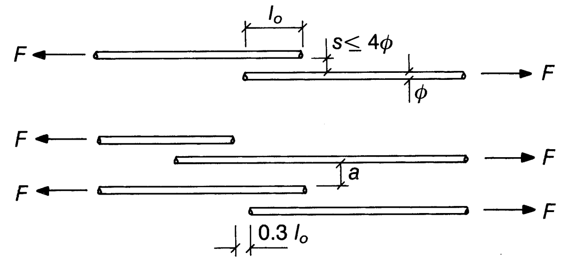


Fig. 9.1.7(b). Staggering and spacing of splices: $s \leqslant 4\phi$; $a \geqslant 2\phi$; $a \geqslant 20\,mm$

- Transverse tensile stresses of concrete should be safely taken by transverse reinforcement when needed (see clauses 9.1.2.2.2 and 9.1.2.2.3).

9.1.2.2.2. Permissible percentage of spliced reinforcement

(*a*) When the provisions of clause 9.1.2.2.1 are respected, the rules of Table 9.1.2 apply for bars in tension.

When transverse reinforcement is provided in excess of the basic requirements (clause 9.1.2.2.3), higher percentages of lapped bars may be used provided that appropriate justification is given.

For secondary reinforcement see clause 9.1.2.3.1b.

Table 9.1.2. Permissible percentage of lapped bars in tension in one section, if transverse reinforcement is not provided

Type of bar	Monotonic load effects	Repeated load effects
High-bond bars		
If only in one layer	100%	100%
If in several layers	50%	50%
Plain bars		
ϕ < 16 mm	50%	25%
$\phi \geqslant$ 16 mm	25%	25%

Secondary ('distribution') reinforcement may be lapped in one section.

(*b*) The percentage of lapped bars in compression in any section may be 100% of the total reinforcement.

Transverse reinforcement is required to resist transversal forces developed by inclined concrete struts assuring the transmission of longitudinal force from one bar to the other.

The amount of the transverse reinforcement can be reduced in the case of transverse compression.

The confinement needed can also be obtained by sufficient concrete mass or by stirrups not necessarily in contact with the lapped bars section.

9.1.2.2.3. Transverse reinforcement in the splicing region

(a) Transverse reinforcement for bars in tension

If the diameter ϕ of the lapped bars is smaller than 16 mm or if the percentage of lapped bars in one section is equal to or less than 25%, then the minimum transverse reinforcement provided for other reasons (e.g. shear reinforcement, distribution bars) is considered to be sufficient.

If $\phi \geqslant 16$ mm or the percentage of lapped bars in one section is more than 25%, then

(i) the transverse reinforcement should have a total area (sum of all transverse bars perpendicular to the layer of the spliced reinforcement) of not less than the area of the spliced bar (Fig. 9.1.8)

(ii) if $a \leqslant 10\phi$ (Fig. 9.1.8), transverse bars should be formed by stirrups (a being the transverse distance between consecutive lap splices, see Fig. 9.1.7b).

The transverse reinforcement should be placed along the two outer sections of the splice ($l_o/3$) (Fig. 9.1.8(a)), between the longitudinal reinforcement and the concrete surface.

(b) Transverse reinforcement for bars permanently in compression

One bar of the transverse reinforcement should be placed outside of each end of the lap length and within 4ϕ of the ends of the lap length (Fig. 9.1.8(b)).

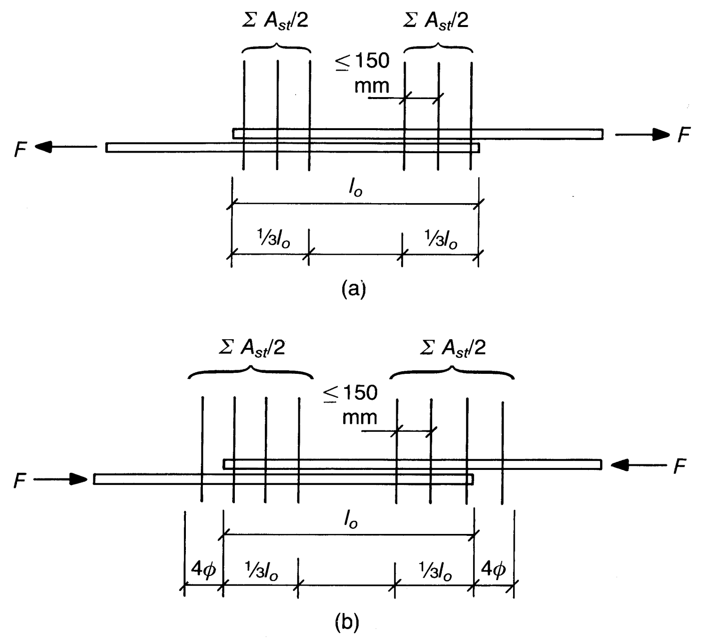


Fig. 9.1.8. Detailing rules and notation for lapped splices (plan view): (a) in tension; (b) in compression

9.1.2.3. Overlapping of welded fabric

9.1.2.3.1. General requirements

Welded fabric can only be spliced by lapping.

(a) Main reinforcement

The splices can be made either by intermeshing or by layering of the fabric (Fig. 9.1.9).

The term 'main reinforcement' is used for reinforcement, irrespective of its direction, when it is provided by calculation to resist action effects (e.g. in two-way spanning slabs).

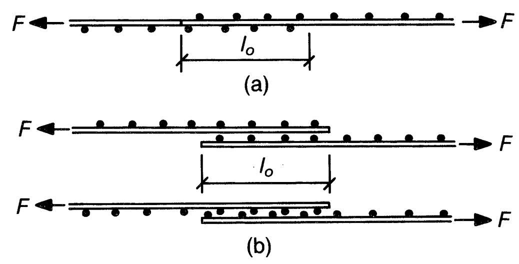


Fig. 9.1.9. Lapping of welded fabric: (a) intermeshed fabric (longitudinal section); (b) layered fabric (longitudinal section)

Where fatigue loads occur, intermeshing should be adopted (clause 1.6.4.2a).

For intermeshed fabric, the lapping arrangements for the main longitudinal bars should conform with clause 9.1.2.2 and subsection 6.9.5 (neglecting the favourable effect of the transverse bars: thus taking $\alpha_2 = 1.0$).

For layered fabric, the splices of the main reinforcement should generally be situated in zones where the design stress of the reinforcement for the ultimate limit state is not more than 80% of the design strength.

Where this condition is not fulfilled, the effective depth of the steel taken into account in the calculations, in accordance with section 6.3, should apply to the layer furthest from the tension face; in addition, due to the discontinuity at the end of splices, when carrying out a crack-verification (Table 7.4.3), next to the joint, the steel stress used in Table 7.4.3 should be increased by 25%.

A reinforcement is called 'secondary' when it is provided transverse to the main steel in order to undertake specific (not calculated) action effects, such as local transverse moments, temperature or shrinkage effects etc. For this reinforcement, the term 'distribution reinforcement' is sometimes used.

(b) Secondary reinforcement

Where the reinforcement acts as 'secondary reinforcement' both arrangements (i.e. wires in the same plane or in different planes) are allowed irrespective of the type of loads (fatigue loads or not).

9.1.2.3.2. Permissible percentages of lapped welded fabric reinforcement

(a) Main reinforcement

For intermeshed fabric, the values given in Table 9.1.2 are applicable.

For layered fabric the permissible percentage of the main reinforcement that may be spliced by lapping in any section depends on the specific cross-sectional area A_s/s of the welded fabric

(i) if $A_s/s \leqslant 1200\,\text{mm}^2/\text{m}$, then 100%
(ii) if $A_s/s > 1200\,\text{mm}^2/\text{m}$, then the welded fabric may be spliced only if it is in an interior mesh of multiple layers. The permissible percentage of bars to be spliced by lapping in one section is 60% of the total reinforcement area.

The joints in multiple layers should be staggered by $1.3l_o$ (l_o from eq. (6.9–8)).

(b) Secondary reinforcement
When the reinforcement acts as 'secondary reinforcement', splices in the transverse reinforcement can be placed in the same cross-section and may extend to the whole (100%) of the total area of the reinforcement in that section.

(c) Laps of the secondary reinforcement
For intermeshed fabric, clause 9.1.2.2.3 applies.

For layered fabric, the length of the lap is chosen from Table 9.1.3.

Table 9.1.3. Required lap lengths for splices of the secondary reinforcement (layered fabric)

Diameter of wires (mm)	Lap lengths
$\phi \leqslant 6$	$\geqslant 150$ mm; at least 1 wire pitch within the lap length
$6 < \phi \leqslant 8.5$	$\geqslant 250$ mm; at least 2 wire pitches
$8.5 < \phi \leqslant 12$	$\geqslant 350$ mm; at least 2 wire pitches

9.1.2.4. Splices by mechanical devices

Mechanical connections of bars are based on couplers.

A coupler is a prismatic or cylindrical steel device which allows the joining of two bars end to end. The force in a bar is transferred to the other bar through the coupler.

9.1.2.4.1. Classification of mechanical connections

Mechanical connections are characterized according to their properties with regard to load transfer: compression-only connections and tension-compression connections.

Connections may need additional materials (e.g. filling materials) or necessitate end preparation of the bars (e.g. threading).

In this category the following types are available

- threaded sleeve
- wedge-locking sleeve
- bolted steel sleeve (solid type or strap type)
- metal-filled sleeve.

The tension-compression connections may be based

- on threading, like
  - steel sleeve threaded for special bars
  - threaded sleeve, rolled threads on bars
  - taper-threaded steel coupler
- on material filled, like
  - metal filled sleeve
  - mortar filled sleeve
- or on sleeve deformation, like
  - forged steel sleeve
  - cold swaged steel sleeve
  - cold swaged steel sleeve with threaded ends.

In accordance with these data, appropriate consequences in design of reinforced concrete should be considered; such as design strength values, allowable redistribution and type of analysis to be used.

It shall be possible to distinguish clearly between

- bars of various grades
- reinforcement that is weldable and that which is not
- type of steel with respect to ductility (see section 2.2).

Special attention, both in design and construction, is needed in cases of simultaneous use of various steels in the same element (e.g. S220 and S400 as main reinforcement in slabs).

(*a*) *Compression-only connections* are based on compressive stress transfer by coaxial contact from one bar to the other. The ends of the bars shall be square cut and in concentric contact. Alternatively, a compression transfer based on a metal-filled sleeve or a threaded sleeve may be applied.

(*b*) *Tension-compression connections* shall be designed for full tension and compression capability and the connection has to meet certain requirements regarding its elongation and ductility properties.

In all cases, the recommendations of appropriate technical approvals should be used.

9.1.2.4.2. Requirements

Technical approvals should specify the following characteristics of connections

- characteristic values of yield and rupture strengths
- deformation properties of the connections
- fatigue characteristics.

9.1.3. Arrangement of the longitudinal reinforcement in a cross-section

9.1.3.1. Simultaneous use of steel of different types

The simultaneous use of steels of various types is allowed on condition that this is taken into account in the design and that no confusion is possible during execution.

Bars spliced by lapping may be in contact along the lap length.
The maximum size of the aggregate should be chosen to facilitate concreting and adequate compaction of the concrete surrounding the bars.

9.1.3.2. Clear distance in the horizontal and vertical direction

The bars in the various horizontal layers should be arranged in vertical planes, leaving sufficient space between them to allow for internal vibration.
The intermediate horizontal or vertical free space between parallel single bars or horizontal layers of parallel bars, should be at least equal to the largest bar diameter but not less than 20 mm.

9.1.4. Additional rules for high-bond bars of large diameter

For high-bond bars of diameter $\phi > 32$ mm, the following rules supplement those given in subsections 9.1.1 and 9.1.2.

Splitting forces are higher and dowel action is greater in case of large-diameter bars; hence reinforcement should be located in the corners of stirrups. A stirrup can surround at most three bars per layer in beams and in slabs where shear reinforcement is necessary.
Where there are more than two layers, the bars situated near the walls of the section should be surrounded by additional stirrups with ends parallel to the planes of the layers of reinforcement

(*a*) When large diameter tension bars are used, a skin reinforcement is needed to keep the crack widths within acceptable limits, unless a verification by calculation is carried out in accordance with section 7.4.

The area of skin reinforcement referable to the skin concrete section $A_{ct,ext}$ (see Fig. 9.1.10) should be not less than

- 0.01 in the direction perpendicular to large diameter bars
- 0.02 parallel to those bars.

The skin reinforcement can be taken into account in the design, provided that it meets the requirements for the arrangement and anchorage of these types of reinforcement.
Skin reinforcement shall have adequate concrete cover (see clause 8.4.3d).

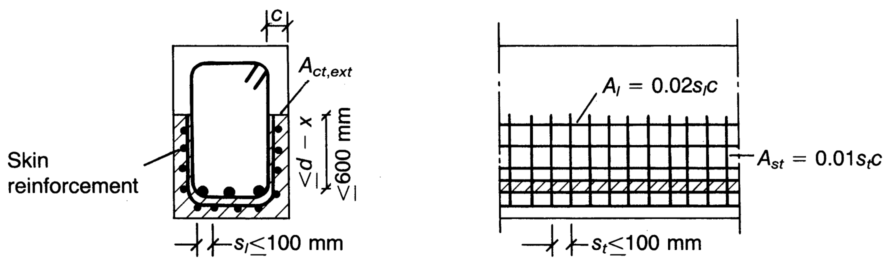


Fig. 9.1.10. Skin reinforcement (x is the depth of the neutral axis at the ultimate limit state)

In the absence of transverse compression, additional transverse reinforcement is required in the anchorage zone (see section 9.1.2). For straight anchorages (see Fig. 9.1.11), the area of this additional reinforcement should not be smaller than

- in the direction parallel to the tension face

$$A_{st} = 0.25\,A_s n_1 \qquad (9.1\text{–}1)$$

- in the direction perpendicular to the tension face

$$A_{sv} = 0.25\,A_s n_2 \qquad (9.1\text{–}2)$$

where

A_s denotes the area of an anchored bar
n_1 is the number of layers with anchored bars in a section
n_2 is the number of anchored bars in one layer.

Transverse reinforcement should be uniformly distributed in the anchorage zone with spacings which should not exceed five times the diameter of the longitudinal reinforcement.

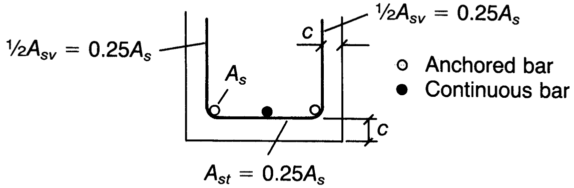


Fig. 9.1.11. Additional reinforcement in an anchorage zone with bar diameter greater than 32 mm and no transverse compression: in this case, $n_1 = 1$, $n_2 = 2$, and $A_{st} = \frac{1}{2}A_{sv} = 0.25A_s$

(*b*) Additional care should be taken in the anchorage zones of large diameter bars which should be anchored as straight bars or through mechanical devices.
(*c*) Lapped splices of large diameter bars are not recommended.

9.1.5. Additional rules for bundled bars

9.1.5.1. General

The number of bars in the bundle should be limited to

- $n \leqslant 4$ for vertical bars in compression and for the bars of a lapped splice
- $n \leqslant 3$ for all other cases.

Sections 6.9 and 7.4 and subsections 9.1.3 and 9.1.4 concerning free space, bond, crack control and skin reinforcement, apply, taking into account the equivalent diameter.

Not more than two bars in contact may be placed in a single plane.

9.1.5.2. Equivalent diameter

For design purposes, bundles of bars containing n bars having the same diameter are replaced by a single notional bar having the same centroid, and an equivalent diameter

$$\phi_n = \phi\sqrt{(n)} < 55\,\text{mm} \tag{9.1–3}$$

For bundles comprising n bars of different diameters, ϕ_n is the diameter of a notional bar with the same area and the same centroid as the bundle of bars under consideration.

9.1.5.3. Minimum concrete cover

The equivalent diameter ϕ_n is taken into account in the evaluation of the minimum cover. However, the cover provided should be measured from the actual outside contour of the bundle of bars.

9.1.5.4. Horizontal and vertical free distances

Subsection 9.1.3 applies, taking into account the equivalent diameter ϕ_n.

It may be necessary to adopt spacings greater than the minimum spacings to allow concreting and the passage of an internal vibrator (especially when a large number of bundles is used), unless special measures are taken to obtain good compaction of the concrete surrounding the bars.

9.1.5.5. Anchorage

The anchorages of the individual bars of a bundle should be straight.

The anchorages should be staggered: for bundles of 2, 3 or 4 bars, the staggering should be respectively 1.2, 1.3 or 1.4 times the anchorage length of the individual bars.

The anchorage length of the whole of the bundle may also be determined on the basis of the equivalent diameter ϕ_n. This method may be used for anchorages over supports.

9.1.5.6. Lapped joints

Joints can be made on only one bar at a time but at any one section. The laps of the individual bars should be staggered in accordance with clause 9.1.5.5.

In the case of laps using sleeved compression joints for bundles, the distance between adjacent sleeves can be reduced to twice the diameter of the sleeve.

9.1.6. Detailing rules for zones of introduction of prestressing forces

The rules presented in this subsection make use of those given in sections 3.3 (local compression), 6.3.4 (longitudinal shear in flanged sections), 6.8 (discontinuity regions), 7.3 (stress limitations) and 7.4 (limit state of cracking).

The quantitative rules of this subsection are meant for anchorages of post-tensioned tendons. They have to be supplemented in the case of couplers. They remain qualitatively valid (needing however some specific supplements) for zones of anchoring of pretensioned tendons and zones of deviation of external tendons.

9.1.6.1. Spreading of the prestressing force

A zone close to any anchorage is a discontinuity region.

It extends on both sides of the anchorage and in all directions.

The extent of this discontinuity zone in front of the anchorage (i.e. conventionally on the same side as the tendon) is larger than (commonly twice) its extent behind the anchorage.

The main part of the stress field in the discontinuity region consists of the spreading of the prestressing force by compression from the anchorage up to the end (in front of the anchorage) of the discontinuity region. A simplified model is based on the assumption that the force generates compressive stresses uniformly spread from the anchorage device within an angle 2β, where $\tan 2\beta = 2/3$ (Fig. 9.1.12).

In fact, $\tan 2\beta$ may range between 2/3 and 1 (see clause 9.1.6.2c).

For a T-beam it is assumed that the dispersion of the prestress is effectuated

- in the middle plane of the web, starting from the anchorage device, within an angle 2β
- in the middle plane of the top slab, if the spreading in the web reaches it, on both sides of the rib, following an angle β.

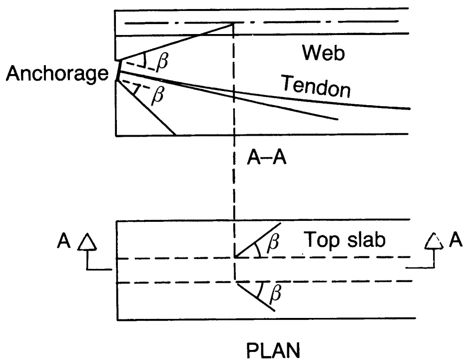


Fig. 9.1.12. Dispersion of prestress

For other shapes of members more detailed models should be used.

For example, box girders.

9.1.6.2. Local analysis of the discontinuity region

(*a*) The spreading of the prestressing force in a field of compressive stresses results, for equilibrium

(i) in tensile spalling and bursting stresses perpendicular to the compressive stresses
(ii) in the case of T-beams, in shear stresses between web and flange.

Other tensile or shear stresses may be necessary for equilibrium for other member shapes.

The location in space of all these stresses may be assessed approximately.
The resultants of these stress fields shall be superimposed on the other action effects according to section 6.3, in order to establish a more complete truss model to be used for the verification of the resistance of the region (ultimate limit state).

(*b*) Appropriate models should be used to control the widths of cracks developed in the discontinuity region due to local compression deformation.

In cases where several anchorages are considered in the same region, specific schemes of cracks due to the effects of these anchorages may occur and, if the crack widths are not controlled, they may result in ultimate failure (see CEB Bulletin 163).

More concentrated prestressing forces (stronger anchorage devices) may necessitate supplementary measures for crack control.

9.1.6.3. Additional reinforcement in the discontinuity region

The limit measures relating to reinforcement are applicable to all the following types of reinforcement.

(*a*) Reinforcement shall be provided and designed for a sufficient resistance (ULS) in all places where tensile or shear internal forces are put in evidence by the model obtained by superimposition according to clause 9.1.6.2(a).

The location of the reinforcement may be found approximately.
Examples are the reinforcement A_{s1} and A_{s2} in Fig. 9.1.13 and a shear reinforcement between web and flange in the case represented in Fig. 9.1.12 (compare also clause 6.9.12.1).

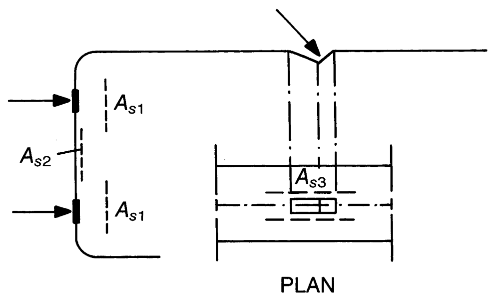


Fig. 9.1.13. Local reinforcement in the vicinity of anchorages of tendons

(*b*) Closely spaced reinforcement is necessary where internal tensile deformations are identified according to clause 9.1.6.2(b), in order to reduce these deformations and spread them over several small-width cracks.

These bars shall be sufficiently long in order to cover the uncertainty of the models used.
Examples are the reinforcements A_{s2} and A_{s3} in Fig. 9.1.13.

(*c*) Sufficient reinforcement is necessary for crack control (ductility requirement) in all other parts of the discontinuity region where it is envisaged that tensile stresses may occur. In all these parts a minimum reinforcement defined by eq. (7.4–16) should be provided.

Because of the high uncertainty of the local analysis, small compressive stresses may in reality turn out to be tensile stresses.

9.1.7. Horizontal and vertical clear distance for internal prestressing steels

9.1.7.1. General

The tendons for internal post-tensioning and pretensioning should be placed, considering simultaneously the sheathing and ordinary reinforcement where relevant, in such a manner that the concrete may be easily placed between them.

9.1.7.2. Post-tensioning

The sheathings shall be located so that

- the concrete can be safely placed without damaging the sheathings
- the concrete can resist the forces from the sheathings in the curved parts under and after tensioning
- no grout will leak into other sheathings during the grouting process.

The minimum horizontal and vertical free spacing for the tendons are given in Fig. 9.1.14.

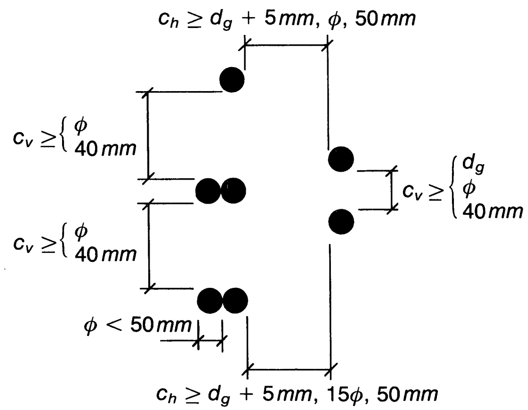


Fig. 9.1.14. Minimum clear spacing for sheathings (where d_g is the maximum aggregate size), see Appendix d.6.5 and ISO 565

When bundles of sheathings are used, the horizontal free spacing between two bundles should be calculated on the basis of the equivalent diameter of bundles.

9.1.7.3. Pretensioning

The minimum horizontal and vertical free spacing of tendons are given in Fig. 9.1.15.

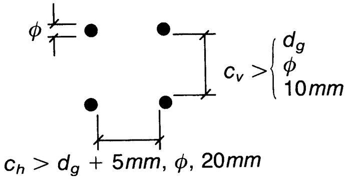


Fig. 9.1.15. Minimum clear spacing for pretension tendons (where d_g is the maximum aggregate size)

Other layouts are acceptable in special cases, provided that test results show satisfactory ultimate behaviour with respect to

- the concrete in compression at the anchorage
- the spalling of concrete
- the anchorage of pretensioned tendons
- the danger of corrosion of the tendons at the end of elements
- the placing of the concrete between the tendons.

9.2. DETAILING OF STRUCTURAL MEMBERS

The quantitative rules of this section are not valid for prestressed elements. However, they are qualitatively valid also for prestressed elements. In case of prefabrication see chapter 14.

9.2.1. Slabs

This subsection applies to rectangular solid slabs, supported by beams and cast in situ, which satisfy the conditions given in section 5.5 and for which the smallest dimension is not less than five times the effective depth.

Slabs may be classified as

- one-way slabs, which are calculated to resist flexural stresses in only one direction
- two-way slabs, which are calculated for flexure in more than one direction.

Conventionally, terms used to describe the geometry (e.g. lateral, horizontal) refer to a slab with a horizontal midplane. The 'top' face of a slab is that on which a load is applied, which is assumed to exert a pressure.

This subsection does not apply to one-way slabs which can be assimilated to beams, i.e. slabs where

- the two free edges are quasi-parallel, at a distance at least equal to three times the total depth
- the main bending moment quasi-parallel to the free edges is much higher than the other main bending moment having the same sign.

9.2.1.1. Main flexural reinforcement

9.2.1.1.1. General

Appropriate minimum reinforcement percentages are needed in order to satisfy several requirements exceeding those covered by calculation.

In general, a slab may be reinforced with four layers (two 'lower' layers and two 'upper' layers). The given rules apply separately to each layer.

The reinforcement is called 'main reinforcement' irrespectively of its direction when it is provided by calculation to resist normal action effects (see section 6.4).

The reinforcement is called 'secondary' reinforcement when it is provided transverse to the main one to undertake specific (not calculated) action effects such as local transverse moments, temperature or shrinkage effects.

In two-way slabs, the reinforcement provided in both directions is considered as 'main' reinforcement; the reinforcement perpendicular to each direction should also satisfy the rules given for secondary reinforcement.

The secondary reinforcement should be provided even if reinforcement is required in one direction only (e.g. flange beams, see clause 9.2.2.3).

The maximum spacing of bars is recommended as follows

- for main reinforcement $s_{max} < 1.2h$ or 350 mm whichever is the less
- for secondary reinforcement $s_{max} < 2h$ or 350 mm whichever is the less

where h denotes the total depth of the slab.

The yield force provided by the secondary reinforcement should not be less than 0.2 times that of the main reinforcement at any section.

For high concentrated loads this ratio should be at least equal to 0.33.

9.2.1.1.2. Partially restrained edges

If the edge of a slab is partially restrained and this restraint has not been considered in the analysis, top reinforcement should be able to balance at least one-quarter of the absolute value of the maximum moment in the corresponding span. This reinforcement should extend from the face of the support over a distance of at least 0.2 times the corresponding span.

Corner lift creates tensions in the 'upper' face which act roughly in the direction of the bisector of the corner angle, and create tensions in the 'lower' face at right angles to that bisector.

9.2.1.1.3. Corner top reinforcement

If the corner of a slab formed by two simply supported edges is prevented from lifting and such restraint is not taken into account in the analysis, and if the existing upper reinforcement is not capable of resisting a moment at least equal to the value of the maximum moment in the span, additional reinforcement should be provided at the corner.

If at the corner, one edge is simply supported and the other restrained, the total top orthogonal reinforcement should be capable of resisting a moment equal to at least one-quarter of the maximum moment in the span.

Restrained corner reinforcement can be omitted if the consequence of cracking in the corner of the slab is not important.

The corner top reinforcement should extend from the face of the support over a distance of at least 0.2 times the smaller span.

Shear reinforcement is not necessary in slabs if $V_{Sd} \leqslant V_{Rd1}$ (see subsection 6.4.2).

9.2.1.1.4. Staggering rule for slabs without shear reinforcement

One half of the maximum area of reinforcement needed in the span should be extended as far as the supports, where it should be suitably anchored.

9.2.1.2. Shear reinforcement

9.2.1.2.1. General

A condition which is analogous to that in clause 9.2.1.1.4 is given in the rule on staggering (see clause 9.2.2.5).

The stirrups should surround the bars (i.e. the top longitudinal reinforcement and the bars of the bottom reinforcement). In general, their slope to the middle plane of the slab should lie between 45° and 90°.

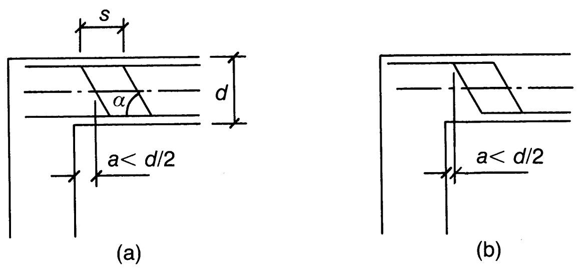


Fig. 9.2.1. Recommended spacing of shear reinforcement: (a) stirrups; (b) bent-up bars

Generally, the angle of bent-up bars to the horizontal should not be less than 30°.

The spacings of the various layers of shear reinforcement should satisfy the condition

$$s \leqslant 0.75d(1 + \cot \alpha) \qquad (9.2\text{–}1)$$

The distance between the face of a support and the nearest layer of shear reinforcement should not exceed $d/2$. That distance is to be taken

- for stirrups, at the middle plane of the slab
- for bent-up bars, at the level of the top reinforcement in bending.

9.2.1.2.2. Zones close to linear supports

The shear reinforcement should be arranged in zones where V_{Sd} is greater than V_{Rd1} (see clauses 6.4.2.3 and 6.4.2.5). A minimum percentage of reinforcement is required.

The transverse spacing of bars in the same layer of shear reinforcement should not exceed $1.5d$ or 800 mm whichever is the smaller.

The shear reinforcement may consist solely of bent-up bars if

$$V_{Sd} \leqslant F_{Rcw}/3$$

where F_{Rcw} is the resistance of web concrete in compression as calculated by eq. (6.3–11).

If not, the stirrups should by themselves satisfy the minimum percentage rule.

9.2.1.2.3. Staggering rule and anchorage lengths of main reinforcement

Clause 9.2.2.5, which gives staggering rule for beams, applies to one-way and two-way spanning slabs.

See subsection 6.4.3.

9.2.1.2.4. Punching shear reinforcement

For punching shear reinforcement account can be taken of

- stirrups contained in a zone at a distance not exceeding $2d$ and 800 mm from the loaded area; the condition resulting from eq. (9.2.–1) should be respected in all directions
- bent-up bars passing above the area defined by a contour line located a distance $d/4$ away from the contour line of the loaded area.

9.2.1.3. Free edges

Along the length of a free edge, a slab should contain (see Fig. 9.2.2)

- reinforcement parallel to the edge consisting of at least two bars, one in the top corner and the other in the bottom corner
- reinforcement perpendicular to the edge, and of which the free ends extend up to a distance of at least $2h$ from the edge.

In thick slabs, the reinforcement along the length of a free edge is to be distributed over the slab thickness.

The reinforcement perpendicular to the edge may consist of U-shaped stirrups enclosing the longitudinal bars along the edge.

The ordinary reinforcement of the slab may form an edge reinforcement.

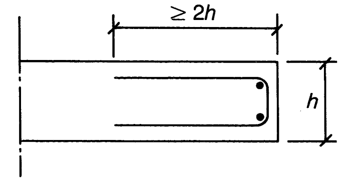


Fig. 9.2.2. Reinforcement along a free edge

9.2.1.4. Hollow or ribbed slabs

The rules given in clauses 9.2.1.1 to 9.2.1.3 can be applied where appropriate.

The top slab of ribbed or hollow block slabs should be reinforced with a mesh providing, in each direction a cross-sectional area not less than 0.001 of the section of the top slab.

If the rib spacing exceeds 1 m, reinforcement in conformity with sub-section 6.3.4 should be provided.

9.2.2. Beams

See commentary to section 9.2.

9.2.2.1. Longitudinal reinforcement

A minimum area of longitudinal bonded reinforcement should be provided to avoid brittle failure in case of unforeseen loss of concrete tensile strength.

If a specific study is not carried out in this respect, the area of longitudinal tensile bonded reinforcement provided should be at least taken equal to

- $0.0015 b_t d$ for steel grades S400 and S500
- $0.0025 b_t d$ for steel grade S220

where b_t is the average width of the concrete zone in tension.

In a T-beam, if the neutral axis in the ULS is located in the flange, the width of the latter is not taken into account in evaluating b_t.

A minimum transverse reinforcement could also be required to prevent buckling of the longitudinal compression reinforcement.

See clause 9.2.3.2.

Appropriate maximum values of tensile reinforcement ratio should be respected in order to ensure a minimum level of ductility.

If a specific study is not carried out in this respect, this maximum ratio may be taken equal to 4% other than at laps.

9.2.2.2. Shear reinforcement

The shear reinforcement should form an angle of 90° to 45° with the axis of the beam.

In most cases, the shear reinforcement in beams consists of vertical stirrups enclosing the longitudinal tensile reinforcement (Fig. 9.2.3) and anchored according to clause 9.1.1.4.

Closed stirrups can be arranged as shown in Fig. 9.2.3. The lap splice of stirrup in the web shown in Fig. 9.2.3(d) is allowed only for high-bond bars, provided that there is no risk of corrosion of the stirrup.

It may also consists of a combination of stirrups and

- bent-up bars
- shear assemblies in the form of cages or ladders of high-bond bars that are cast in without enclosing the longitudinal reinforcement (see Fig. 9.2.4). Their anchorage should comply with the provisions of clause 9.1.1.4 (see Fig. 9.1.6(c) and (d).

Attention is drawn to the risk of splitting cracks to open when the longitudinal bars, which are near the outer faces of elements, are bent-up.

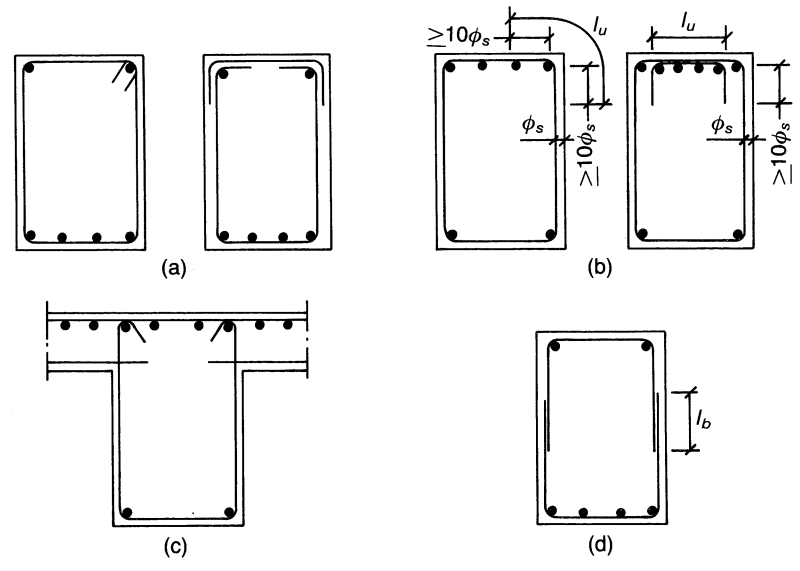


Fig. 9.2.3. Possible layout of stirrups: (a) closing in compression zone; (b) closing in tensile zone; (c) closing in compression zone; (d) this detail has been used but does require particularly good workmanship to achieve its design purpose

For bent-up-bars, the minimum mandrel diameter should be not less than the values given in Table 9.1.1.

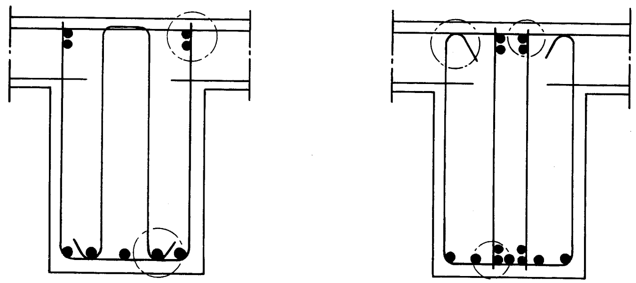


Fig. 9.2.4. Examples of combination of stirrups and shear assemblies: for detailing of the anchorages in the circled zones, see Fig. 9.1.6

The values given in Table 9.2.1 should be taken as minimum reinforcement percentage.

$$\rho_w = \frac{A_{sw}}{s b_w \sin\alpha} \qquad (9.2\text{–}2)$$

See clause 6.3.3.1.

Except for the cases listed in clause 6.3.3.1 any beam should have a minimum number of ties or stirrups over its entire length. This minimum reinforcement is required to ensure that the failure load is higher than the cracking load.

Smooth shear reinforcement bars should not exceed 12 mm in diameter.

The maximum spacing s_{max} of various layers of shear reinforcement is defined by the following conditions

$$s_{max} = 0.7d < 300\,\text{mm} \quad \text{for } F_{Scw} \leqslant \tfrac{1}{5}F_{Rcw}$$

$$s_{max} = 0.6d < 300\,\text{mm} \quad \text{for } \tfrac{1}{5}F_{Rcw} < F_{Scw} \leqslant \tfrac{2}{3}F_{Rcw}$$

$$s_{max} = 0.3d < 200\,\text{mm} \quad \text{for } \tfrac{2}{3}F_{Rcw} < F_{Scw}$$

where F_{Scw} is the acting compression force of web concrete (see eq (6.3–10)) and F_{Rcw} is the resistance of web concrete in compression as calculated by eq (6.3–11).

The transverse spacing of legs in each group should not exceed $2d/3$ or 800 mm, whichever is smaller.

Table 9.2.1. Minimum reinforcement values of ρ_w in percent

Concrete	Minimum reinforcement [%]		
	steel S220	steel S400	steel S500
C12	0.15	0.08	0.06
C20	0.20	0.11	0.09
C30	0.26	0.15	0.12
C40	0.32	0.18	0.14
C50	0.37	0.21	0.16
C60	0.42	0.23	0.18
C70	0.46	0.26	0.20
C80	0.51	0.28	0.22

9.2.2.3. Beam flanges

For the area of transverse reinforcement (tying a beam flange to the web) calculated in accordance with subsection 6.3.4, a minimum reinforcement is normally required.

See subsection 6.3.4 for the calculation of this reinforcement. The minimum percentage of the total reinforcement crossing the connection, calculated by eq. (9.2–2) replacing b_w by the thickness of the top slab, can be taken equal to the values in Table 9.2.1.

9.2.2.4. Torsional reinforcement

Clauses 9.2.2.1 and 9.2.2.2 also apply to the longitudinal bars and stirrup of beams subjected to torsion, except the stirrup arrangements of Fig. 9.2.4 which are not allowed in the case of torsion.

The values indicated in clauses 9.2.2.1 and 9.2.2.2 can be adopted for the torsional reinforcement as well.

The spacing of stirrups should not exceed a value $u_{ef}/8$, where u_{ef} denotes the length of the perimeter of the stirrups.

For notation, see subsection 6.3.5.

The longitudinal bars shall be placed so that at least one bar is located in each corner of the stirrup, the other bars being distributed uniformly along the internal perimeter of the stirrups, at a spacing not exceeding 350 mm.

9.2.2.5. Curtailment of bars

The tensile envelope should be calculated from eq. (6.3–4). Beyond the section, where the reinforcement is no longer required to carry the full force, the reinforcement can be curtailed with an anchorage length of $l_{b,net}$ + 100 mm.

Since shear forces are taken into account when calculating the tensile chord forces (section 6.3), there is no need for any additional 'moments' staggering rule.

For the definition of $l_{b,net}$ see Fig. 6.9.6.

Unless the section of zero moment is exactly known, a horizontal range equal to the possible variation range of the zero moment shall be assumed.

9.2.3. Columns

For some other limit measures, see chapter 10.

9.2.3.1. Longitudinal reinforcement

(*a*) Unintended eccentricities and the need for controlling creep deformations leads to the requirements of a minimum percentage of longitudinal reinforcement.

If a more specific study is not carried out in this respect, a value equal to $0.008A_c$ may be used.

(*b*) In order to ensure easy compaction of concrete and safe splicing of longitudinal bars, an appropriate maximum steel percentage should be respected.

In the absence of a specific study on the matter, the area of the longitudinal reinforcement should not exceed $0.04A_c$, with the exception of regions of lap splicing where it can reach $0.08A_c$.

(*c*) The minimum number of longitudinal bars is four for rectangular columns and six for circular columns.

The diameter of longitudinal bars should preferably not be less than 12 mm.

9.2.3.2. Transverse reinforcement

Generally, the transverse reinforcement consists of stirrups surrounding the longitudinal reinforcement.

The diameter of stirrups shall not be less than 5 mm or one-quarter of the maximum diameter ϕ_l of the longitudinal bars. Their spacing

- shall secure longitudinal compression bars against local buckling
- shall ensure that stirrup legs intersect at least one possible shear crack under the most adverse condition.

If a more specific study is not carried out, the following values may be used

$$\text{spacing } s = 12\phi_l$$
$$s = \min\{h_c, 300\,\text{mm}\}$$

where h_c denotes the smallest dimension of the column.

Under certain circumstances, it may be necessary either to increase the stirrup diameter or to decrease spacing, to prevent bursting or any local secondary damage; special precautions are to be taken

- in zones located on either side of a beam or a slab, over a height equal to the larger dimension of the column section
- at changes of direction of the longitudinal bars.

The transverse reinforcement shall be so arranged that each bar or group of bars placed in a corner, and one of every two intermediate bars of the outer layer of reinforcement are held. Bars can be considered to be held if they are not located at a distance of more than 150 mm from a held bar.

Correct lateral tying of circular columns can be achieved with the aid of hoops or helices around the longitudinal bars or groups of bars.

All transverse reinforcement (ties, stirrups or helices) should be appropriately anchored (see clause 9.1.1.4).

9.2.4. Reinforced concrete building walls

This subsection deals with reinforced concrete walls of which the length measured horizontally is at least equal to four times the thickness.

For some other limit measures, see chapter 10.

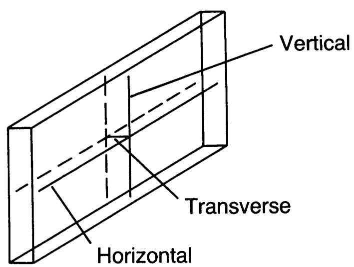


Fig. 9.2.5. Terminology of wall reinforcement

9.2.4.1. Vertical reinforcement

The area of the vertical reinforcement shall lie between $0.004\,A_c$ and $0.04\,A_c$, where A_c is the required sectional concrete area.

In general, half of this reinforcement should be located at each face.

The distance between two adjacent vertical bars should not exceed twice the wall thickness or 300 mm, whichever is lesser.

9.2.4.2. Horizontal reinforcement parallel to the wall faces

Horizontal reinforcement running parallel to the faces of the wall (and to the free edges) should be provided and arranged at each surface. Its minimum section should not be less than 30% of that of the vertical reinforcement. The spacing between two adjacent horizontal bars on the same surface should not be greater than 300 mm. The diameter should not be less than one-quarter of that of the vertical bars.

The necessary reinforcement for the control of cracking is given in section 7.4.

9.2.4.3. Horizontal reinforcement perpendicular to the wall faces (transverse reinforcement)

If the area of the load carrying vertical reinforcement exceeds 0.02 A_c then clause 9.2.3.2 applies.

9.2.5. Deep beams

9.2.5.1. Simply supported deep beam on two supports

9.2.5.1.1. Longitudinal reinforcement

The main longitudinal reinforcement corresponding to the ties considered in the design model should be uniformly distributed over a depth measured from the lower face of the beam of about $0.12h$ or $0.12l$ whichever is lesser where

- h is the total height of the beam
- l is the design span.

It should be fully extended from one support to the other.

At supports, the anchorage should be obtained by using horizontal hooks or U-loops or by anchorage plates, unless the length between the support and the end of the beam is greater than the anchorage length of $l_{b,net}$ (see subsection 6.9.1).

Because of the danger of splitting, anchorage by means of vertical hooks should be avoided.

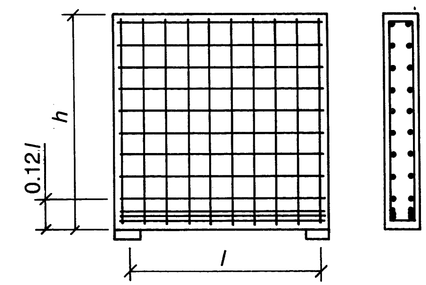


Fig. 9.2.6. Distribution of longitudinal reinforcement in a deep beam in case of direct loading (schematic)

9.2.5.1.2. Additional reinforcement

(a) Direct loading (the load is applied on the top of the beam)

In this case, an additional reinforcement in the form of a mesh of

orthogonal reinforcement consisting of a horizontal layer surrounded by a vertical layer should be arranged.

The total percentage of the bars in each direction should not be smaller than 0.2% (i.e. 0.1% for each face).

(b) Suspended loading (the load is applied at the bottom of the beam)

In this case, the orthogonal mesh described in (a) above should be supplemented by additional stirrups to transmit the total load between its application level and the level corresponding to the lesser of h and l.

The reinforcement should surround the bars of the main reinforcement and be fully extended over a depth equal to the lesser of h and l.

Near the supports the height of the stirrups may be slightly reduced (by about 20%).

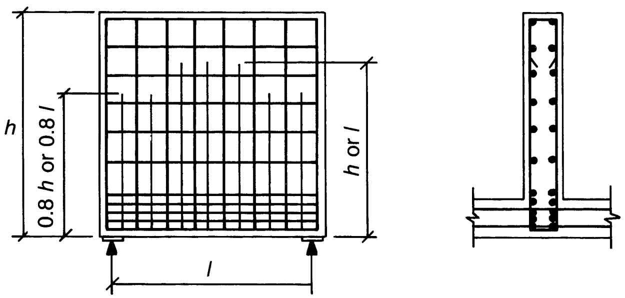


Fig. 9.2.7. Recommended reinforcement layout for suspended loading (schematic)

(c) Vertically distributed load

According to the chosen design model, the force transmitted to the deep beam should be resisted by an additional reinforcement (suspension reinforcement) made either of vertical stirrups extended without cut-off near the common volume over a length equal to the lesser of h and l (Fig. 9.2.8(a)), or by bent-up-bars (adequately anchored) resisting about 60% of the load placed symmetrically to the line of action of the load, and by complementary stirrups (Fig. 9.2.8(b)).

This case corresponds to a load applied over the total depth of the beam by means of a transverse perpendicular wall or by a column of large cross-section which is extended down the lower part of the beam.

For the minimum diameter of the curvature of the bars, see Table 9.1.1.

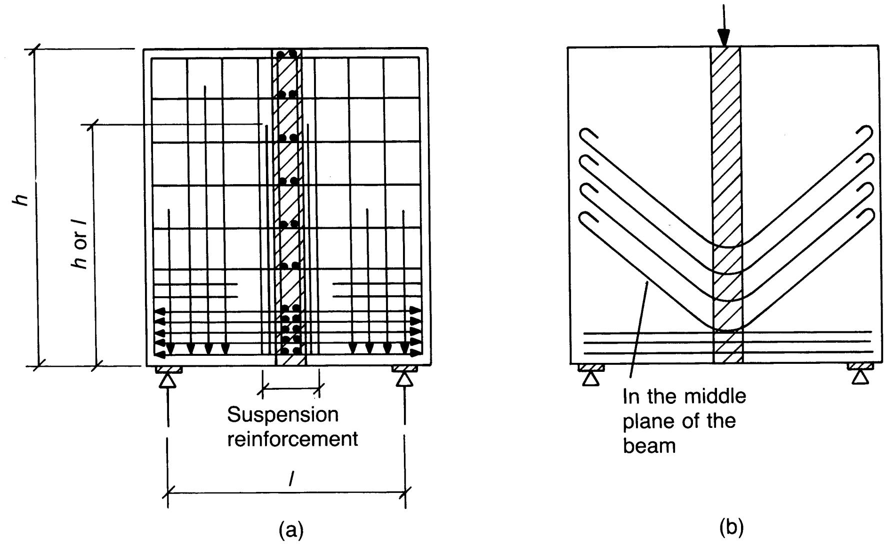


Fig. 9.2.8. Reinforcement for vertically distributed load

9.2.5.2. Continuous deep beams

(a) In the span (i.e. in the positive moment area)

For the main reinforcement as well as for shear reinforcement, clause 9.2.5.1 applies.

(b) Over the supports

For the main horizontal tensile reinforcement

- a fraction $[(l/h) - 1)]/2$ of the total required cross-sectional area of reinforcement should be placed in the upper strip which extends to $0.2h$ or $0.2l$ whichever is the less (see Fig. 9.2.9)
- the remaining cross-sectional area should be uniformly distributed within the lower strip just below, which extends to $0.6h$ or $0.6l$, whichever is the less (see Fig. 9.2.9)

Where $h > l$ supplementary longitudinal reinforcement should be placed near the upper face of the beam.

- one bar on two may be stopped symmetrically at a distance from each face of the support equal to $0.4h$ or $0.4l$, whichever is the less.

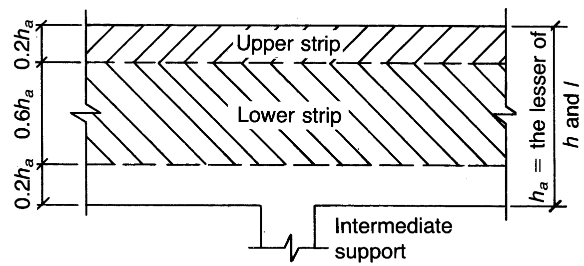


Fig. 9.2.9. Distribution of reinforcement over the supports

10. LIMIT MEASURES

10.1 INTRODUCTION

This chapter contains information related to required 'limit measures' (i.e. minimal and/or maximal rules) regarding

- the materials to be used
- the dimensions of concrete cross-sections and covers
- the cross-sections and arrangement of steel.

This information is mainly a summing up of relevant clauses from all the Model Code chapters, completed in some cases with guidance based on previous experience. Further limit measures may be needed for other cases not covered by this Code.

These limit measures may have been dictated by several conditions and requirements such as

(*a*) conditions for the validity or the avoidance of calculations
(*b*) minimum ductility needed
(*c*) qualitative reliability and insensitivity requirements
(*d*) functional demands
(*e*) durability
(*f*) friendliness of execution.

It should be noted, however, that a limit measure may serve, simultaneously, more than one of the above listed conditions and requirements.

The application of the Code is only possible if all these conditions are appropriately considered by the designer (see also subsection 1.1.5), even in case this Model Code does not foresee explicit limit measures for all of them.

On the other hand, when such explicit measures are foreseen, the designer may also make use of alternative means to face the above mentioned six conditions or requirements, provided that full evidence is available on the matter.

This chapter intends to assist the designer in the first step of design, i.e. in conceptual design (before a detailed analysis and verification are performed). After the selection of a structural system, dimensions have to be preliminarily given to concrete and eventually steel; thus, stiffnesses may be determined for the analysis, and verifications may be carried out at the end.

These preliminary dimensions are given out of the experience of the designer; however, a certain guidance may be offered by summing up all information related to 'limit measures' required throughout the Code. These limit measures may have to do with the selection of the quality of materials to be used, as well as with concrete dimensions and steel sections; all may influence predimensioning as well as final verifications.

Limit measures (see subsection 1.1.5) may have been foreseen in a code for the following possible reasons.

(*a*) The validity of the engineering models or of the practical rules suggested for design may be secured only under conditions regarding the quality of materials or the dimensions of structural elements (e.g. maximum distance of stirrups in beams, maximum concrete class, conditions of release of calculating secondary reinforcement, etc.). In the same category belong the conditions allowing for the avoidance of some calculations.

(*b*) A minimum level of ductility is needed, even in cases this fundamental property is not explicitly checked; and such minimum ductility may need certain limit measures (e.g. maximum steel ratio) in order to be implemented.

(*c*) Qualitative reliability and insensitivity requirements may dictate some minimal measures (e.g. a minimum thickness of elements or the need for at least two tendons available in each concrete section).

(*d*) Functional requirements may also be covered by minimal measures (e.g. the non-damageability of non-structural elements is secured by restricting the deformability of slabs).

(*e*) Practical durability measures may directly concern minimum covers, minimum bar-diameters, minimal concrete classes, etc.

(*f*) Friendliness of execution may dictate minimum values of thickness of concrete sections, maximum steel percentage, etc. Finally, some inspectability considerations may influence dimensioning.

This is mainly the case for those measures which are not compulsory.

10.2. QUALITY OF MATERIALS

10.2.1. Concrete grades

(*a*) A minimum concrete grade is specified to ensure the validity of models and to ensure good durability.

(i) Only grades C16 and above shall be used for reinforced concrete (clause 2.1.1.2).

(ii) Only grades C25 and above shall be used for prestressed concrete (clause 2.1.1.2).

(iii) For plain precast wall panels ($\rho < 3‰$) grade C16 and above shall be used, while for reinforced precast wall panels ($\rho \geqslant 3‰$) grade C20 and above shall be used (clause 14.2.5.6).

For compression joints (precast, subsection 14.3.2) the strength of joint mortar material should not be less than 70% of the adjacent precast concrete strength (if the mortar is not transversely confined), unless the joint is designed accounting for actual mortar strength.

For segmental construction, the compressive strength of the material of wide joints should be not less than 25 MPa or that of the joined segments (clause 14.7.1.3), while the compressive strength of the material of match joints should be at least equal to that of the concrete of the joined segments (clause 14.7.1.4).

(*b*) Concrete constitutive laws and models should be used with caution for $f_{ck} > 50$ MPa (clause 2.1.1.1); the relation between shear resistance and concrete strength depends (for higher concrete grades) upon the characteristics of the aggregates (clause 6.4.2.3).

In addition, an upper concrete grade should be foreseen taking into account the conditions of minimum ductility and ease of execution. In this Model Code this limit is C80.

10.2.2. Reinforcing steel

(*a*) Grades higher than S500 require further consideration concerning the validity of the given rules, i.e. $\sigma - \varepsilon$ diagrams, bond, etc. (clause 2.2.4.2).

(*b*) Regarding ductility, necessary whether or not moment redistribution is taken into account (clause 2.2.4.4), three classes are foreseen

class A: $(f_t/f_y)_k \geqslant 1.08$, $\varepsilon_{uk} \geqslant 5.0\%$
class B: $(f_t/f_y)_k \geqslant 1.05$, $\varepsilon_{uk} \geqslant 2.5\%$
class S: $(f_t/f_y)_k \geqslant 1.15$, $\varepsilon_{uk} \geqslant 6.0\%$.

Adequate margins of safety and low local damageability are ensured by these minimal measures.

To be used where high ductility of structural elements is required (i.e. in seismic regions).

In seismic regions, additional requirements for Class S may be introduced, e.g. in order to reduce uncertainties in applying the capacity design criterion.

$$(f_{yk})_{act}/(f_{yk})_{nom} \leqslant 1.30$$

10.2.3. Prestressing steel

(*a*) Grades

Lower limiting values for the characteristic tensile strength of prestressing steels are dictated by the applicability of prestressing, where upper limiting values are imposed by the possibilities of production of prestressing steels.

Other conditions are foreseen as well, regarding reverse bending behaviour, constriction, etc.

(*b*) Regarding ductility (clause 2.3.4.4), the unit elongation at maximum load (ε_{uk}) shall be at least equal to 0.035.

*Table 10.2.1. Limit measures concerning quality of materials**

	Validity of models/ calculations	Ductility requirements	Reliability requirements	Functional demands	Durability requirements	Execution requirements	Reference
Concrete grade							
Minimum	RC: C16 PC: C25 Precast: C16/C20	—	•	—	•	—	2.1.1.2 14.2.5.6
Maximum	C50	•	—	—	—	•	2.1.1.1 6.4.2.3
Reinforcing steel							
Max. f_{yk}	S500	—	—	—	—	—	2.2.4.2
Ductility	—	3 classes $\varepsilon_{uk} \geqslant \ldots$ $(f_t/f_y)_k \geqslant \ldots$	—	—	—	—	2.2.4.4
Prestressing steel							
Max. f_{pyk}	—	—	—	—	—	—	—
Ductility	—	$\varepsilon_{uk} \geqslant 0.035$ and other conditions	—	—	—	—	2.3.4.4

*Among the rules not explicitly mentioned in this Code, the most important ones of which the designer should take note are marked by a circle.

In clauses 6.4.2.3 and 14.4.2.1 minimum lengths are foreseen for steel bars beyond the centre-line of supports as well as for bearing of floors made of hollow-core units.

10.3. CONCRETE DIMENSIONS

10.3.1. Support widths

For concrete elements (slabs and beams) supported by concrete walls and columns or by masonry, minimum support widths should be foreseen, in order to ensure safe and reliable transfer of forces and to prevent anchorage failure of the main longitudinal reinforcement.

10.3.2. Span to depth ratios

If the deflections of slabs or beams are not numerically checked, the height of the flexural element should follow the minimal rule

$$d \geqslant l/\lambda$$

where λ is taken from Table 7.5.2 and eq. (7.5–5) of clause 7.5.2.3.

This rule intends to ensure low damageability of non-structural elements.

10.3.3. Slenderness

The dimensions of the cross-section of vertical structural elements may be influenced by slenderness considerations if second order effects are to be disregarded. The corresponding limit value of slenderness is given in subsection 5.4.2. See also clause 6.6.3.1.

10.3.4. Minimum dimensions

It is assumed that the minimum thickness of concrete in any part of a linear element is

- 80 mm for reinforced concrete and prestressed concrete (subsection 6.3.1)
- 50 mm for precast concrete (clause 14.2.5.3).

In addition the minimum thickness of beams cannot be less than five times the maximum size of aggregates or five times the diameter of sheathing or tendons.

However, use of such extremely small thicknesses should be made only under special conditions securing against unforeseen loading or environmental conditions.

Minimal geometrical conditions are given

- in clause 14.4.2.2 for composite floors with precast slabs
- in clause 14.4.2.3 for composite floors with ribs and blocks
- in clause 14.2.5.6 for precast wall panels
- in clause 14.7.1.4 for joints in precast/segmental structures.

10.3.4.1. Stress limitations

Excessive compressive stress in the concrete under service load may lead to longitudinal cracks in the concrete or higher levels of creep. If the proper functioning of a member is likely to be adversely affected by these, measures should be taken to limit the stresses to an appropriate level.

For further details see section 7.3.2 and clause 1.6.6.2.

(*a*) If the stress does not exceed $0.6 f_{ck}(t)$

(i) under the rare combination, longitudinal cracking is unlikely to occur
(ii) under the quasi-permanent combination, creep and the corresponding prestress losses can be correctly predicted.

(*b*) If under the quasi-permanent combination the stress exceeds $0.4 f_{ck}(t)$, the non-linear model shall be used for the assessment of creep (see clause 2.1.6.4.3d).

(*c*) Durability requirements for prestressed concrete may impose other limits on stresses in the concrete, e.g. that the section should remain in compression (see clause 7.3.1.1).
In addition, creep effects may also be checked at transfer zones in prestressed beams.

10.4. CONCRETE COVER

Cover minimal values may be required for several reasons, e.g. as a measure to secure

Where fire resistance is needed, other limits may apply.

- steel-to-concrete bond resistance
- protection of steel against external corrosive agents
- ease of concrete pouring.

Similarly, in the absence of additional skin reinforcement, maximal values may be needed in order to limit surface cracking and spalling.

Depending on the requirement under consideration, the appropriate cover values should be retained.

(*a*) To ensure that bond forces are safely transmitted and to prevent spalling of the concrete, the maximum cover of any bar, tendon or sheathing of diameter ϕ should be at least equal to ϕ (section 9.1).
In case of bundled bars, the equivalent diameter ϕ_n is taken into account in evaluating the minimum cover. However, the cover provided should be measured from the actual outside contour of the bundle (clause 9.1.5.3).

The equivalent diameter ϕ_n (< 55 mm) is calculated according to clause 9.1.5.2.

When anchoring is made by means of bends, hoops or loops, it is recommended that in the anchorage area, the thickness of the cover would be equal to 3ϕ (see also clause 9.1.1.2b).

This rule is also applicable in the case of beam top bars bent inside the column in a beam-column connection.

(*b*) To ensure good durability (section 8.4), the minimum distance between any concrete surface and the nearest reinforcement bar or the sheathing for such tendons, shall be obtained from Table 8.4.1.

Table 10.4.1. Minimum cover, c_{min}

Exposure class	c_{min} (mm)
1	10
2	25
3, 4	40
5	*

*Depends on the individual type of environment encountered.

The values are absolute minimum values with no downward tolerances allowed and no upward tolerance being specified.

The nominal values c_{nom}, are equal to the minimum values plus tolerance according to the following rule:

$$c_{nom} = c_{\min} + \text{tolerance}$$

tolerance = 10 mm with normal quality control
tolerance = 5 mm with intensified quality control.

Reduction of concrete cover is related to better compaction in precast units than those specified in Table 8.4.1.

Where fire resistance is needed, other limits may apply.

For precast elements, nominal values of concrete cover may be reduced by 5 mm with respect to the values specified in Table 8.4.1, if accurate control on bar position and on concrete compaction is carried out.

However, depending on local conditions, lower limit values of cover thickness may lead to the need of additional skin reinforcement.

(*c*) To prevent cracking and spalling of thick concrete cover, in cases enhanced durability is desired (subsection 8.4.6) or high strength/high bond and large diameter bars are used ($\phi > 32$ mm, subsection 9.1.4), special skin reinforcement shall be provided if $c_{nom} \geqslant 70$ mm.

The arrangement of information in these tables makes clear that several requirements do not always lead to a limit measure. However, for some of those 'empty' cases, several National Codes do foresee respective rules.

10.5. STEEL CROSS-SECTIONS AND ARRANGEMENTS

In Tables 10.5.1 to 10.5.5 limit provisions mainly for reinforcing steel are summarized separately for each category of building elements.

*Table 10.5.1. Limit measures concerning concrete dimensions/covers**

	Validity of models/ calculations	Ductility requirements	Reliability requirements	Functional demands	Durability requirements	Execution requirements	Reference
Concrete dimensions							
Minimum support widths	•	—	•	—	—	—	—
Max. l/d (for buildings)	—	—	—	$\lambda_o k_T k_t \left(\frac{400}{f_{yk}}\right)$	—	—	7.5.2.3
Columns max. λ	6.6.3.1	—	—	—	—	—	5.4.2 6.6.1.3
Minimum dimensions: beams	RC, PC: 80 mm Precast: 50 mm	—	—	—	—	$5d_{aggr}$, 5ϕ	6.3.1 14.2.5.3
slabs	•	—	•	—	•	—	—
columns	•	—	•	—	—	•	—
walls	—	—	•	—	—	•	—
Compressive stress limit	$0.4f_{ck}$ creep	—	—	—	$0.6f_{ck}$ longitudinal cracks	—	7.3 1.6.6.2
Cover							
Minimum	ϕ bond	—	—	—	10–(40) mm	$2d_{aggr}$	9.1 8.4
Maximum	—	—	—	—	For $c_{NOM} \geqslant 70$ mm skin reinforcement	—	8.4.6

*Among the rules not explicitly mentioned in this Code, the most important ones of which the designer should take note are marked by a circle.

*Table 10.5.2. Slabs**

	Validity of models calculations	Ductility requirements	Reliability requirements	Functional demands	Durability requirements	Execution requirements	Reference
Shear reinforcement (if needed)							
Minimum mechanical ratio ($\omega_{sw} = A_{sw}f_{yk}/(b_w s f_{ctm})$	$\geqslant 0.20$	—	—	—	—	—	6.4.2.4
Spacing	$< 0.75d\,(l + \cot\alpha)$	—	—	—	—	—	6.4.2.4
Inclination to the horizontal							
stirrups	$\alpha \geqslant 45°$	—	—	—	—	—	6.4.2.4
bent-up bars	$\alpha \geqslant 30°$	—	—	—	—	—	6.4.2.4
Longitudinal reinforcement							
Maximum bar diameter related to the steel stress (under service loads)	—	—	—	See Table 7.4.3	See Table 7.4.3	—	—
Maximum bar spacing (in a cross-section) related to steel stress (under service loads)	—	—	—	See Table 7.4.4	See Table 7.4.4	—	—
Minimum reinforcement area within the tensioned zone	$A_{s,\min} = k_c k f_{ct\max} A_{ct}/\sigma_{s2}$ (see eq. (7.4–16))	—	—	—	—	—	—
Tensile stresses in steel (under the rare combination)	$< 0.8 f_{yk}$ $< f_{yk}$ (when stresses are due only to imposed deformation)	—	—	—	—	—	7.3.3
Clear distance in the horizontal or vertical direction	—	—	—	—	—	$\geqslant$ largest bar dia. $\geqslant 20$ mm	—
Main flexural reinforcement							
Minimum bar diameter (mm)	—	•	—	—	•	—	—
Maximum bar spacing (mm)	—	Min (1.2h, 350 mm)	—	—	—	—	9.2.1.1.1, lhs
Secondary flexural reinforcement							
Ratio of secondary to main reinforcement:							
distributed loads	$\geqslant 0.20$	—	—	—	—	—	9.2.1.1.1
concentrated loads	$\geqslant 0.33$	—	—	—	—	—	9.2.1.1.1
Maximum bar spacing (mm)	Min (2h, 350 mm)	—	—	—	—	—	9.2.1.1.1, lhs

* Among the rules not explicitly mentioned in this Code, the most important ones of which the designer should take note are marked by a circle.

*Table 10.5.3. Beams**

	Validity of calculations	Ductility requirements	Reliability requirements	Functional demands	Durability requirements	Execution requirements	Reference
Longitudinal reinforcement (bending)							
Minimum bar diameter (mm)	•	—	•	—	•	—	—
Minimum number of bars (or tendons)	•	—	•	—	—	—	—
Minimum percentage of reinforcement: $\rho_s = A_s/(b_t d)$							
S400, S500	—	0.0015	—	—	—	—	9.2.2.1, lhs
S220	—	0.0025	—	—	—	—	
Maximum percentage of reinforcement	—	0.040 (outside zones of splices)	—	—	—	—	9.2.2.1
Maximum bar diameter related to the steel stress (under service loads)	—	—	—	See Table 7.4.3	—	—	—
Minimum reinforcement area within the tensioned zone	—	—	—	See eq. (7.4–16)	—	—	—
Tensile stresses in steel (under the rare combinations of actions)	—	—	—	$< 0.8 f_{yk}$ $< f_{yk}$ (when stresses are due only to imposed deformation)	—	—	7.3.3
Maximum bar spacing (within a cross-section) related to steel stresses	—	—	—	See Table 7.4.4	—	—	—
Clear distance in the horizontal and vertical direction	—	—	—	—	—	⩾ Largest bar dia.	9.1.3.2
						⩾ 20 mm	9.1.3.2
Longitudinal reinforcement (torsion)							
Minimum bar diameter (mm)	$S/16$, where S is spacing of stirrups	—	—	—	—	—	6.3.5.2
Minimum reinforcement	One bar at each corner of the stirrup + bars uniformly distributed along the internal perimeter of the stirrup at a spacing < 350 mm	—	—	—	—	—	9.2.2.4
Transverse reinforcement (shear)							
Minimum bar diameter	—	—	—	—	—	—	—
Maximum bar diameter	12 mm (for smooth bar)	—	—	—	—	—	9.2.2.2

Table 10.5.3. cont

	Validity of calculations	Ductility requirements	Reliability requirements	Functional demands	Durability requirements	Execution requirements	Reference
Minimum percentage $\rho_w = A_{sw}/(sb_w \sin\alpha)$	—	See Table 9.2.1					
Minimum mechanical ratio of stirrups $\omega_{sw} = A_{sw}f_{yk}/(b_w s f_{ctm} \sin\alpha)$	$\geqslant 0.20$	—	—	—	—	—	6.4.2.4
Inclination of transverse reinforcement to the axis of the member	$\geqslant 45°$ (stirrups) $\geqslant 30°$ (bent-up bars)	—	—	—	—	—	6.4.2.4
Spacing at stirrups longitudinal	—	—	Depending on value of F_{Scw}/F_{Rcw}	—	—	—	9.2.2.2
transverse	—	—	Min $\{2d/3, 800\,\text{mm}\}$	—	—	—	9.2.2.2
Transverse reinforcement (torsion-closed stirrups)							
Minimum bar diameter (mm)	—	—	—	—	—	—	9.2.2.4
Minimum percentage of reinforcement	—	—	—	—	—	—	9.2.2.4
Maximum spacing longitudinal	Min $\{0.5b, 0.5d\}$ $< u_{ef}/8$, where u_{ef} is the perimeter of the stirrup	—	—	—	—	—	9.2.2.4
transverse	$0.75d$	—	—	—	—	—	6.3.5.4

*Among the rules not explicitly mentioned in this Code, the most important ones of which the designer should take note are marked by a cross.

Table 10.5.4. Columns

	Validity of models/ calculations	Ductility requirements	Reliability requirements	Functional demands	Durability requirements	Execution requirements	Reference
Longitudinal reinforcement							
Minimum number of bars:							
rectangular columns	—	—	4	—	—	—	9.2.3.1
circular columns	—	—	6	—	—	—	9.2.3.1
Minimum bar diameter (mm)	12	—	—	—	—	—	9.2.3.1, lhs
Minimum percentage of reinforcement	—	—	0.008	—	—	—	9.2.3.1
Minimum area of reinforcement	$A_s = 0.15 N_{Sd}/f_{yd}$	—	—	—	—	—	6.6.2.4
Maximum percentage of reinforcement (in zones of splices)	—	0.08	—	—	—	—	9.2.3.1
Transverse reinforcement							
Minimum bar diameter (mm)	$\geqslant$ max. $\{\phi_t/4, 5\,\text{mm}\}$, where ϕ_t = diameter of longitudinal bars	—	—	—	—	—	9.2.3.2
Maximum bar diameter (mm)	—	—	—	—	—	—	9.2.3.2
Maximum spacing of stirrups (mm)	—	—	$12\phi_{t,\min} b$, 300 mm, where b is smallest dimension of concrete section	—	—	—	9.2.3.2
Minimum spacing (mm)	—	—	—	—	—	—	9.2.3.2
Maximum distance of longitudinal bars not held by a stirrup	—	—	—	150 mm in plane (buckling)	—	—	9.2.3.2

Table 10.5.5. Walls

	Validity of models/ calculations	Ductility requirements	Reliability requirements	Functional demands	Durability requirements	Execution requirements	Reference
Vertical reinforcement							
Minimum bar diameter	—	—	—	—	—	—	9.2.4.1
Minimum percentage of reinforcement ρ_v	—	—	0.004	—	—	—	9.2.4.1
Maximum percentage	—	0.04	—	—	—	—	9.2.4.1
Maximum spacing of bars	Min $\{2t, 300\,\text{mm}\}$ where t is wall thickness	—	—	—	—	—	9.2.4.1
Horizontal reinforcement							
Minimum bar diameter	—	—	—	—	—	—	9.2.4.2
Minimum percentage of reinforcement	—	—	$\geqslant 0.30\rho_v$	—	—	—	9.2.4.2
Maximum percentage	—	—	—	—	—	—	9.2.4.2
Maximum spacing of bars	$\not> 300\,\text{mm}$	—	—	—	—	—	9.2.4.2
Transverse reinforcement (for $\rho_v > 0.02$)	—	—	—	—	—	—	9.2.4.3

PART III. CONSTRUCTION AND MAINTENANCE

11. PRACTICAL CONSTRUCTION

11.1. GENERAL

This section is essentially drafted from the designer's viewpoint to ensure that the assumptions behind the design requirements are not invalidated in construction. It includes a series of minimum specification requirements for the standard of workmanship.

In essence the designer specifies the requirements for materials and execution for the type of structure to be built. Such requirements are to be given on drawings and in specifications. It is the task of the contractor to check the given instructions, to implement them, and to ensure in his work the required level of quality.

The content is not extensive enough to be used as a contract document. More information for carrying out construction (including safety on site) should be taken from manuals such as the FIP Guide to Good Practice —Basic reinforced concrete and prestressed concrete construction, FIP London, 1978.

Durability and service life aspects are treated in section 8.1.

In the implementation of the material in this chapter, an appropriate quality assurance plan as in chapter 12 should be adopted.

11.2 SITE

11.2.1. General

Responsibilities should be clearly defined. Decision and information charts should be established at the beginning of the work.

Referring to concrete construction, the following fields of activities may appear

- representative of the owner
- project manager, overall responsible for design and execution
- resident engineer, representing the design manager
- site manager, who is responsible for all activities on the site
- contractor, who is responsible for the execution of the work in general
- sub-contractor for specific work, e.g. related to prestressing
- supplier of materials
- co-ordinator for design and construction in a particular field, e.g. falsework and centring.

In ordinary construction where standard practice is ensured, these charts may not be needed.

It should be noted that the list of activities is an example; in some countries other definitions may be specified.

These activities may be assumed by different persons (in case of large or complicated structures); some of them may be assumed by the same person depending on the size of the work and accepted standard practice.

11.2.2. Project management

The following tasks may pertain to the project management

- co-ordination of design, construction and maintenance; definition of responsibilities; official filing

- overall construction programme including commencement of works, excavation, falsework, erection sequences
- communication between design and construction, including inspections and feedback, in particular
  - — to control completeness of designer's documents for construction as drawings, material characteristics, reinforcement lists, specifications
  - — to inform designer about construction documents elaborated from site engineers, about actual soil conditions and site records.
- Relevant Quality Management Systems and Quality Assurance should be considered.

11.2.3. Site management

The following tasks may pertain to the site management

- check of basic design assumptions
- organization of control procedures on site
- minutes of co-ordination meetings
- records of site operations
- as-built documentation.

Examples of design assumptions to be verified

- soil conditions
- temporary excavation
- centring design.

11.2.4. Preparation work for site

The check list of the preparation of the site should include

- existing installations
- excavation
- site equipment
- erection procedures
- working places and their protection
- safety on site.

For detailed information refer to: Bygghälsan Report 'Survey of Working Environment in Concrete Construction', FIP Stockholm, 1982; FIP Guide to Good Practice 'Prestressed concrete—Safety precautions in post-tensioning', Thomas Telford, London, 1989.

11.2.5. Inspections

The inspection should cover materials, records, workmanship and construction.

Tests concerning reinforcement, constituent materials and production of concrete should be defined.

Modifications of the structural design and the specifications should not be introduced except with the approval of the designer or of the supervisory authority.

For detailed information refer to CIB/FIP/CEB Manual on formwork technology 'Coffrage', CIB Rotterdam, 1985.

For bridge construction, the following check list is recommended: Traggerüste, Check-Liste, Ingenieurbüro Krebs und Kiefer, Darmstadt, 2nd edition 1982.

11.3. FORMWORK, FALSEWORK AND CENTRING

For major projects a co-ordinator for design and construction of falsework and centring should be nominated.

11.3.1. Basic requirements

(*a*) Give shape to concrete structure within the prescribed tolerances (see Chapters 1 and 4).
(*b*) Give desired surface texture.
(*c*) Allow thorough placing of reinforcement, tendons and concrete (and its compaction).
(*d*) Allow space for prestressing jacks.
(*e*) Resist actions during construction, including those due to prestressing (equipment, deformations during tensioning).
(*f*) Allow striking without damage (see section 11.9).

11.3.2. Design

(*a*) *Principles*. Design of falsework and centring to be based on codes with a safety concept equal or comparable to the present Model Code for Concrete Structures.
(*b*) *Actions*

- (i) Vertically: permanent and variable loads during construction; wind (uplift); support reactions due to prestressing, especially in construction stages (e.g. displacement of dead load effects during tensioning of tendons).
- (ii) Horizontally: pressure of fresh concrete (depending on method of placement), effect of included formwork, wind (bracing), swing of crane load (accidentally), local deviation force in curved pipes for concrete pumping.
- (iii) Dynamic action: impacts (e.g. due to waves), vibration (compaction of concrete).
- (iv) Effects of ground settlements.
- (v) Thermal effects (concrete hardening, heat treatment).

(*c*) *Choice of structure*. Should be rather stiff to reduce deformations during erection; should allow unrestricted deformation during tensioning of tendons. Special attention to be given to proper (ground) supports and joints (e.g. mortar bed).

Checks can be made by calculation, testing or accepted standard practice.

Use of standardized material according to manufacturer's instructions and approval documents.

(*d*) *Structural calculation*. Verify resistance (load-bearing capacity) and deformations (sufficiently stiff to ensure that limits of tolerances for concrete structures are satisfied) with special attention to the compatibility with concrete operations (concreting programme) and to the interaction between concrete and falsework during tensioning of tendons (if necessary, remove part of falsework, e.g. to avoid 'springing-up' due to resilient effect of falsework). Camber according to design calculations of the concrete structure and designer's drawings.

(*e*) *Tightening of formwork*. Prevent harmful loss of material during concreting (proper joints, if necessary, to be sealed).

(*f*) *Formwork remaining in concrete structure*. Check durability and compatibility with concrete; if it is acting as a structural element, ensure stability and correct anchorage.

(*g*) *Formwork spacers left in concrete structure*. Should not impair durability and appearance.

(*h*) *Accelerating curing*. Ensure uniform distribution of temperature through the mass of concrete.

(*i*) *Cooling equipment for concrete in structures with large dimensions*

(*j*) *Striking* (see section 11.9). Formwork, falsework and centring to be so designed that they can be correctly struck without damaging the concrete structure (if necesary, provide devices for lowering the centring).

11.3.3. Erection

(*a*) *Principles*. Ensure compliance with prepared drawings and specifications (including surface finishes). Not to be removed until sufficient strength of concrete (see section 11.9).

(*b*) *Supports and structural joints*. Ensure correct positioning and force transmission.

(*c*) *Assembly of formwork panels*. Joints should be mortar-tight.

(*d*) *Preparing formwork faces*. Surfaces in contact with concrete should be clean (all rubbish to be removed). Approved mould-release agents to be applied in a thin uniform layer without contaminating the reinforcement; on vertical or sloping surfaces the material should have sufficient

adherence. Concrete to be placed soon after application of the release agent.

(*e*) *Temporary works inserts*. Temporary works inserts may be necessary to assist in keeping formwork, reinforcement, prestressing ducts and other similar items in place until the concrete has hardened. Such inserts should not introduce unacceptable loading into the structure, should not react harmfully with the constituents of the concrete, the reinforcing or prestressing steel and should not produce unacceptable features.

(*f*) *Checks before and during concreting*. See section 11.6 and chapter 12.

11.3.4. Re-use of material

Proper maintenance in order to be equivalent to new products; if used successively, the reduced bearing capacity should be taken into account.

11.4. REINFORCEMENT

11.4.1. Transportation and storage

Steel bar reinforcement, welded mesh reinforcement and prefabricated reinforcement cages should be so transported and stored, that they do not suffer any damage (mechanical or corrosion).

The surface condition of the reinforcement should be examined prior to use to ensure that it is free from mechanical damage (e.g. notches) and from surface deposits damaging bond properties. Overall reduction of the section through corrosion may be tolerated within certain permissible limit values; pitting is not allowed.

For detailed information refer to: CEB Manual on Concrete Reinforcement Technology, Georgi, St-Saphorin (CH), 1983; CEB Bulletin 164 'Industrialization of Reinforcement in Reinforced Concrete Structures', CEB Lausanne, 1985.

11.4.2. Identification

If means of identification has been lost, acceptance tests on samples should be required.

11.4.3. Cutting and bending

Reinforcing steel should be cut and bent in accordance with accepted standard practice.

Bending should be carried out by mechanical methods, at constant speed without jerking, with the aid of mandrels, so that the bent part has a constant curvature.

11.4.4. Welding

Welding shall not be carried out on reinforcing steel unless the steel is suitable for welding (see clause 2.2.5.3).

In general, reinforcing bars should not be welded at or near bends of the bars.

Where a risk of fatigue exists, the welding of reinforcement should conform to special requirements.

The production and check of the welded connections should comply with the relevant requirements.

11.4.5. Joints

The length and position of lapped joints should be in accordance with the design and the drawings.

Joints made with mechanical connecting devices should be carried out in accordance with the specified standards or approval documents.

11.4.6. Assembly

The assembly of the reinforcement should be robust enough to ensure that the bars do not shift from their prescribed position in the course of transport, placing and concreting.

11.4.7. Placing

The specified minimum cover to the reinforcement should be maintained by the use of approved chairs and spacers.

The reinforcement should be secured against any displacement and the position of the reinforcement should be checked before concreting. Tolerance limits are given in clause 1.4.5.2.

In areas of congested reinforcement sufficient spacing should allow for insertion of needle vibrators.

This point should already be implemented during design and detailing of the reinforcement.

11.5. TENDONS

11.5.1. Prestressing steel (transportation and storage)

The following items should be observed

- use of clean containers; suitable protection against environmental influences in case of water transport

Refer also to the forthcoming FIP report on 'Corrosion protection of prestressing steel (pretensioning, post-tensioning and stay cables)'.

- protection (during transport to the site and while on site) against mechanical damage, detrimental corrosion and deposits damaging the bond properties
- storage under solid cover (against rain, ground, aggressive atmosphere) and with ventilation
- no welding close to stored steel without protection from splashes
- periodic examination on site
- very thin film of rust may be accepted if it can be removed by a dry cloth.

The detrimental influence of rust on the friction behaviour should be taken into account.

11.5.2. Sheathing (ducting)

In addition to subsection 11.5.1, the following should be considered

— check aspect (no local damage), no corrosion inside, use of end caps
— leak-proof, water tightness
— sheathing should resist mechanical and chemical attack
— empty ducts (steel inserted later): immediately after concreting, the free passage in the sheathing should be checked, e.g. by pulling-through a conical gauge (dolly).

Empty ducts should be sufficiently rigid or should be stiffened during construction by a temporary mandrel.

11.5.3. Anchorages, couplers

The requirements of the approval documents should be checked.

11.5.4. Fabrication of tendons

(*a*) *General*. Tendons can be fabricated on-site or off-site in permanent installations. In both cases the procedures shall ensure that the final product meets the requirements.

(*b*) *Identification*. Maintain identification of all materials. If identification cannot be established, acceptance tests on samples have to be performed.

(*c*) *Cutting*. Normally by shear, saw or abrasive disc. If by oxy-acetylene flame, excess length will be necessary (use sufficient oxygen to ensure cutting, rather than melting).

(*d*) *Bending of bars*. Not allowed in vicinity of anchorages or couplers. Reverse bending not allowed.

(*e*) *Assembly*

(i) Materials to be checked before use, especially after long storage period (identification, not wet, etc.).

(ii) Fabrication area protected (clear of ground and under cover, the latter if required on account of environmental conditions). Prestressing steel for a tendon, from the same delivery. Anchorage plate perpendicular to tendon axis.

(iii) Sheathing to be jointed by couplers which are taped at both ends for watertightness. Joints in adjacent sheathings to be staggered at least 300 mm apart.

In case of large duct diameters ($\geqslant 80$ mm), near to the peak points, additional holes for later secondary injection should be provided.

(iv) Provide vents (with possibility to be closed) at both ends (threaded for connection to pump), peak points and all points where air or water may accumulate, especially in long tendons (maximum interval 40 m), vertical tendons and significantly inclined tendons.

(*f*) *Transportation.* To be transported and handled with care. Not to be dragged across the ground. When lifting by crane, avoid local crushing and restrict bending.

(*g*) *Placing* (tolerance limits: see section 4.1)

(i) Careful inspection of sheathings before and after placing, and particularly at construction joints.

(ii) Accurate fixing according to designer's specifications (drawings) of dimensions (and tolerances), spacers and supports; prevent empty ducts from floating; ease of casting concrete.

(iii) Secure position of sheathings containing prestressing steel, the use of welding is not allowed for corrugated sheathings; it can be authorized, when sheathing consists of steel tubing. During placing of tendons in the formwork the minimum radii of prestressing steel and sheathing should be respected. All vents and other openings of the tendons shall be closed and marked immediately after placing of the tendon.

(iv) Pull-in or push-in of strands and wires should be made with proper equipment by using procedures which are not harmful to the prestressing steel and the ducts.

(v) Prevent flow of grout between crossing ducts (if insufficient concrete between ducts) by metal strips or other barriers, especially when high grouting pressure is used.

11.5.5. Temporary protection of tendons

In dry environments, the given periods may be extended.

The period between assembly and grouting should be kept to a minimum: in a non-corrosive environment maximum 12 weeks, but not more than 4 weeks inside the formwork (unstressed) and 2 weeks after tensioning.

Use of products only with strict respect of producer's instructions for the application. When water-soluble products are used as temporary protection, the bond properties should not be affected harmfully.

Otherwise corrosion protection should be provided, e.g. by sealing, blowing out the ducts with pre-dried air, use of vapour phase inhibitors or emulsifiable oils.

Suitable protection for threaded ends of bars should be provided.

Protective wrapping should be chemically neutral.

11.5.6. Unbonded tendons

FIP Recommendations 'Design of flat slabs in post-tensioned concrete', May 1980 and 'Corrosion protection of unbonded tendons', Thomas Telford, London 1986.

The use of unbonded tendons in practical construction generally does not differ from practice with bonded tendons. Therefore the relevant sections of this chapter remain valid. It is, however, essential that sufficient care is taken during execution so as not to damage the protection system of such tendons.

11.5.7. External tendons

FIP Recommendations 'External prestressing' (under preparation).

The use of external tendons in practical construction generally does not differ from practice with bonded tendons inside the concrete. Therefore the relevant sections of this chapter remain valid.

11.6. CONCRETE

11.6.1. General

Refer to Appendix d on concrete technology.

11.6.2. Measures to be taken before concreting

Measures to be taken in case of unforeseen stops in concreting or freezing etc. should be duly planned.

The concreting dates shall be fixed well in advance to allow control of geometry, block-outs, inserts, reinforcement and tendons (checking of space for needle vibrator).

Shortly before casting, uncoated timber formwork shall be watered and the position of the void formers checked. Empty ducts should be secured against uplift.

11.6.3. Concreting programme

The following items should be considered in the concreting programme

- concrete pour areas, timetable (sequence of pours)

- construction joints: location, surface
- particular types of surface finish.

Examples: roughing measures, use of retarders.

11.6.4. Measures to be taken after concreting

The fresh concrete should be protected against premature drying out.

Manuals for extreme weather conditions:

FIP Guide to Good Practice—Concrete construction in hot weather, Thomas Telford, London, 1986; STUVO/FIP Report 'Concrete in hot countries', STUVO, 's-Hertogenbosch (NL), 1985; RILEM Recommendations for winter concreting, RILEM Bulletin, Paris, 1963.

11.7. TENSIONING OF TENDONS

11.7.1. General

To secure a correct and safe tensioning of tendons it is necessary to provide

- skilled personnel, trained for the purpose
- solid and safe equipment (regular calibration)
- written instructions and pre-arranged prestressing programme
- suitable safety precautions during prestressing.

11.7.2. Instructions to the site

(a) Basic information
- (i) post-tensioning system, tendon units and prestressing equipment; anchorages and couplers used
- (ii) minimum concrete strengths prior to stage and final tensioning
- (iii) tensioning from one or both ends (at the same time or at one end after the other); order of stressing successive tendons
- (iv) sequences of successive stages of stressing tendons and striking falsework, if such stages are envisaged
- (v) expected values for pressure in the jack, force to be developed at the jack, elongation of steel, draw-in at non-jacking end and slip of steel during anchoring
- (vi) any tests to be performed (e.g. friction).

(b) Detailed programme
- (i) increments in the stressing force of each tendon, including influence of striking falsework
- (ii) corresponding steel elongations, including effects of immediate and time-dependent losses

(iii) permissible deviations from expected elongations should be indicated
(iv) corrective measures to be taken, if the above deviations are not respected.

Suggested values are: 15% for a particular tendon, but not more than 5% for the sum of all values of tendons in the same section.

(c) Methods of measurement

(i) pressure
(ii) force
(iii) elongation
(iv) draw-in or slip.

Refer to the FIP State of the Art Report: 'Tensioning of tendons: force–elongation relationship', Thomas Telford, London, 1986.

(d) Records

(i) observations to be recorded during tensioning
(ii) site stressing records to be transmitted to and signed by the design engineer in view of any necessary adaptation of above instructions.

11.7.3. Tensioning operations

(a) Before stressing

(i) confirm that above instructions can be met
(ii) check concrete strength
(iii) check that structure is free to move
(iv) check that vent holes are not blocked
(v) inspect exposed tendon ends and surfaces on which anchorages and stressing equipment will bear
(vi) calibrate prestressing devices.

(b) During stressing

(i) stresses should increase at a gradual and steady rate; for long elongation: temporary anchoring and regripping
(ii) tensioning log with all data and anomalies observed
(iii) full records of pressures at jack or load cell and of steel elongations corresponding to each increment of the tension force, including temporary overstress, release, draw-in or slip at the ends
(iv) compare measured and calculated values
(v) observe instructions, if deviation is greater than permitted
(vi) maintain tendons to allow eventual restressing; cut-off and grouting only after final approval.

(c) After stressing

- (i) visual inspection of concrete and anchorages
- (ii) supervision of possible temporary protection of tendons
- (iii) for trimming surplus tendon material, apply clause 11.5.4(c) on cutting of tendons.

Where surplus tendon material is burned off, the anchorage devices should be protected from excessive heat development.

11.7.4. Temporary protection after tensioning

A temporary protection may be necessary, if grouting takes place more than two weeks after tensioning, unless it is demonstrated that other measures effectively prevent corrosion.

Protective material shall allow later sufficient bond and shall have no deteriorating effect on the grout. Only approved products should be used.

Refer also to the forthcoming FIP report on 'Corrosion protection of prestressing steel (pretensioning, post-tensioning and stay cables)'.

Use of products only with strict respect of producer's instruction for the application.

11.8. GROUTING OF TENDONS

11.8.1. General

For detailed information refer to the FIP Guide to Good Practice 'Grouting of tendons in prestressed concrete', Thomas Telford, London, 1990.

(a) The main objectives are

- (i) to prevent corrosion of the prestressing steel
- (ii) to provide an efficient bond between the prestressing steel and the concrete.

(b) Basic requirements

- (i) all voids in ducts and anchorages should be filled with a suitable grouting material (usually cement grout)
- (ii) the above objectives are met by using approved grout materials (remain alkaline, no harmful components) and by filling the ducts and anchorages completely, including voids between tendons, with a grout which after hardening fulfils structural requirements (strength, bond)

(c) subsection 11.7.1 (General on tensioning) applies for grouting as well.

11.8.2. Cement grout

(a) Properties

- (i) adequate fluidity and cohesion when plastic
- (ii) low bleed, and shrinkage compensated when hardening
- (iii) adequate strength and resistance to freezing when hard.

(b) Materials

(i) ordinary Portland cement or, in particular cases, other cements
(ii) water
(iii) admixtures: plasticizing agents, viscosity modifying agents, gas generating admixtures, retarders (for long parts of ducts to be grouted)
(iv) chlorides from all sources should not exceed 0.1% by mass of cement.

Other cements than Portland should be used only if their suitability has been verified by tests.
The use of silica fume or similar material is in development, but its benefits have not yet been confirmed.

(c) Mixing

(i) all materials to be batched by mass
(ii) water–cement ratio should not exceed 0.40
(iii) admixtures according to manufacturer's instructions
(iv) general procedure: firstly water in mixer, add cement, after thorough mixing (to get homogeneous colloid) add other materials
(v) time of mixing according to manufacturer's instruction: not more than four minutes; after mixing grout should be kept in continuous agitation, until pumping into the duct (within 30 minutes of mixing, unless retarder is used); temperature of fresh grout should not exceed 35°C.

Lower values than 0.40 are recommended (refer to the above FIP Guide) but in special cases, higher values, up to 0.45, may be used.

11.8.3. Instructions to the site

(a) Preconditions

(i) equipment operational (including 'standby' grout pump to avoid interruptions in case of malfunction)
(ii) permanent supplies of water under pressure and of compressed air
(iii) materials batched (excess to allow for overflow)
(iv) ducts free of harmful material (e.g. water, ice)
(v) vents prepared and identified
(vi) preparation of control tests for grout
(vii) in case of doubt, grouting trial on representative ducts.

(b) Grouting programme

(i) characteristics of grout (including periods available for use, hardening time)
(ii) characteristics of equipment (including pressures, injection rates)

(iii) blowing of air through the ducts, or clearing by methods defined in the approval documents
(iv) order of injection operations and fresh grout tests (fluidity, segregation)
(v) manufacture of test specimens (bleeding, shrinkage, strength)
(vi) grout volume to be prepared
(vii) instructions in case of incidents (e.g. fault during injection: remove grout from duct and repeat injection) and harmful climatic conditions (e.g. after and during periods with temperature lower than 5°C).

In hot climate, alternatively, it may be advisable to flush water through the ducts prior to starting the grouting operation. However, the consequences of remaining water on the water–cement ratio should be considered.

If measuring equipment (temperature gauges) is available, the 5°C value may be referred to the temperature of the structure in the vicinity of the tendon.

(c) Records to be established on all operations, measures, tests and particular events; to be transmitted to the design engineer.

11.8.4. Grouting operations

(a) Before injection, check preconditions and confirm that grouting programme can be fulfilled.

For details refer to the above FIP Guide.

(b) During injection

(i) process to be carried out at a continuous and steady rate (sufficiently fast to prevent segregation at points where flow is restricted; but slow enough in corrugated ducts to prevent entrapping of air in downward stream of grout)
(ii) grouting shall commence at the lowest grouting point (or from the lowest cable end) and continue until the fluidity or density of the grout flowing from the free ends and the vent openings is about the same as that of the injected grout; after closing the last vent, pressure to be held at 0.4 to 1.0 MPa for some minutes
(iii) in some circumstances (large diameter of vertical or inclined ducts, at the highest points of the ducts) post-injection is necessary to replace bleed water by grout
(iv) after completion of grouting, losses of grout from the duct should be prevented
(v) to allow expansion of grout during hardening and to displace bleed water, appropriate vents may be opened.

Grouting of a duct should be done without interruption. The speed at which the grout is pumped through the duct can vary from 5 to 15 m/min. The maximum grouted length should not exceed 120 m.

(c) After injection

(i) all equipment to be thoroughly washed very shortly after grouting, followed by thorough draining of pump, mixer and pipelines

(ii) if large voids are suspected, take precautions for regrouting
(iii) in case of doubt, control with an endoscope or with vacuum.

11.8.5. Sealing

Once the grout has hardened, all openings, grouting tubes and vents have to be hermetically sealed to prevent penetration of water and harmful products (e.g. anti-freeze or de-icing agents).

To obtain a good seal of anchorage recesses, a preparatory treatment to the surface around the anchorages may be applied; in large recesses, connection reinforcement should be provided.

11.8.6. Other protection

Refer also to the forthcoming FIP report on 'Corrosion protection of prestressing steel (pretensioning, post-tensioning and stay cables)'.

Tendons may be protected by materials based on bitumen, epoxy resins, rubber, etc., provided that the effects on bond and fire resistance are not important.

11.9. STRIKING

11.9.1. General

Formwork, falsework and centring should remain undisturbed until concrete has achieved sufficient strength to withstand the stresses and deformations to which it will be subject (with an acceptable margin of safety).

Special consideration should be given to

(a) the weight of the concrete (especially if this is the major part of the full design load)
(b) imposed loads (e.g. from falsework placed for higher elements, before their hardening)
(c) sequence of striking, eventual temporary jacking and temporary support
(d) the need of certain elements being retained for reducing time-dependent deformations (e.g. auxiliary props) or for the stability of the whole structure (e.g. wind-bracing)
(e) prestressing and grouting operations (see sections 11.7 and 11.8)
(f) particular striking operations (e.g. at re-entrant angles of formwork)
(g) environmental conditions (e.g. freezing) and measures available to protect the concrete once the formwork is removed
(h) subsequent surface treatment requirements.

Striking operations to be executed without shock (e.g. by sudden removal of wedges) and by using only static forces.

Supporting elements (columns, walls) should be struck first.

Lowering of centring by respecting the structural system and the prestressing programme.

11.9.2. Minimum periods before striking

The minimum periods before striking depend upon several influences such as strength development, curing, deformation behaviour or dead load ratio.

For prestressed concrete structures striking may be done after tensioning of tendons, which is related to a required minimum strength of concrete.

In the absence of other information the following periods are recommended for reinforced concrete structures, if normal hardening cement is used.

Table 11.9.1. Recommended minimum periods before striking

	Surface temperature of concrete			
	$\geqslant 24°$	16°	8°	2°
Vertical formwork	9 h	12 h	18 h	30 h
Slabs				
soffit formwork	3 days	4 days	6 days	10 days
props	7 days	10 days	15 days	25 days
Beams				
soffit formwork	7 days	10 days	15 days	25 days
props	10 days	14 days	21 days	36 days

The above values for vertical formwork demand that striking is immediately followed by appropriate curing or protection from low or high temperatures.

If, during the hardening of concrete, freezing periods occur, the above values are to be increased, at least by the duration of these periods; for winter concreting in general, refer to the RILEM Recommendations for winter concreting, RILEM Bulletin, Paris 1963.

The values in Table 11.9.1 may have to be increased, if special consideration is given to the limiting of early cracking (especially in elements with different thicknesses or temperatures), or to the reduction of creep deformations.

The values in Table 11.9.1 may be reduced if accelerated curing methods or special formwork (e.g. sliding forms) are used.

12. QUALITY ASSURANCE AND QUALITY CONTROL

Due to practical and historical reasons, the terminology used in this chapter is not in full accordance with the terms defined in ISO 8402—'Quality—Vocabulary' from 1986.
For example, in ISO 8402

- quality assurance plan is denoted quality plan
- quality control means the operational techniques and activities used to fulfil requirements for quality
- inspection is the term used for activities such as measuring and testing.

12.1. QUALITY ASSURANCE

12.1.1. Quality assurance requirements

The subsequent description covers the phases detailed design, production and execution. The preliminary phases (design brief, basic engineering) and the operation and maintenance phases are not covered.
In this chapter only the requirements related to a specific project are specified.

Requirements of a company's quality system are not covered. It is anticipated that each company has established and implemented a quality system covering its basic organization.

12.1.2. Quality assurance plan

For the whole building process, the quality assurance plan (as well as the set of individual quality assurance plans) should at least include, with the appropriate degree of detail, the following elements as suitable for the tasks to be undertaken.

The content of a quality assurance plan depends on the extent of the quality system in the basic organization. The described quality assurance plan assumes that a full quality system is established and implemented in the basic organization. Deliveries and services of sub-contractors shall be covered with reference, where relevant, to their quality assurance plans.

(*a*) *Organization.* The responsibility, authority and the interrelation of all personnel who manage, perform and verify work affecting quality should be defined. The names and qualifications of the personnel responsible for tasks requiring special skill and experience should be stated.

External organization and key personnel involved should be defined.
Internal and external lines of communication should be defined.

(*b*) *Planning.* It should be described how the taks undertaken will be planned in order to achieve a systematic and orderly performance.

(*c*) *Design control.* Procedures to

(i) identify input requirements
(ii) perform the design
(iii) verify the design
(iv) control design changes

should be established and maintained.

The verification measures may include

- design reviews
- discipline check of individual documents
- alternative calculations
- qualification testing.

(*d*) *Document control.* Establish measures to ensure that all essential quality related documents are reviewed and approved by competent and authorized personnel prior to issue.

The control should ensure that the right documents are at the right places at the right time.

(*e*) *Procurement.* Establish measures to ensure that services procured, products purchased and items subcontracted are in accordance with specified requirements.

(*f*) *Production and construction.* Establish measures to ensure that production and construction activities are performed under controlled conditions.

This may include appropriate controls of materials, equipment, processes and procedures, computer software, personnel and associated supplies, utilities and environments.

The inspection, testing and supervision activities necessary to fulfil the specified requirements should be defined.

All inspection, testing and supervision activities should be documented.

(*g*) *Records.* Performance quality records to be maintained as evidence that the service provided and the items delivered/constructed meet contractual or other applicable technical requirements. These records should be delivered to the client or preserved for an agreed period after completion of the project.

12.2. QUALITY CONTROL

12.2.1. Classification of control procedures

Basically two types of control are distinguished and each of them may be performed at various levels.

Although distinguished, the two kinds of control are normally interconnected.

Other distinctions of control methods can be made (e.g. visual or test-based, statistical or total). Refer to Bulletin 191.

12.2.1.1. Types of control

Types of control are

- *production control*, which relates to production and execution processes and includes preliminary tests where relevant; production control applies to materials, components and production and execution activities at least

- *compliance control*, which relates to the results of production and execution processes; it is applicable to all stages of the building process, i.e. to promoting, design, materials and components, production and execution, and use of the completed structures.

Compliance control is normally concluded by the acceptance of products relating to the corresponding stage.

12.2.1.2. Control levels

According to the body which performs a control or is responsible for it, distinction is made between internal and external control.

Internal control may be carried out either by the producer himself, or by another body acting for the producer.

An internal control made by the producer may be performed by the production team or by a specialized service of the producing firm. In case of certified production, a high degree of independence of the controlling body is of paramount importance (see clause 12.2.2.3).

Internal control relates to a production control and possibly (in the case of certified production) to a compliance control, final acceptance excluded.

External control may be carried out either by the client himself, or by another body acting for the client.

Here a contractor is normally considered as the client with regard to his suppliers and subcontractors.

External control is necessary to conclude to a final acceptance of products. In the case of reliable certified production the external compliance control can be radically reduced (merely up to an identification of the product and checking the certificate).

12.2.2. Control systems

12.2.2.1. General

Control systems are rational combinations of production controls and compliance controls. They shall be in accordance with the following rules and subsections 12.2.3 to 12.2.6.

The choice of an appropriate control system for a particular project depends on several parameters. The total control system shall be consistent with the quality measures defined in the quality assurance plan of the project.

The main parameters are

- technical capacity, availability and degree of independence of the participants
- current practice concerning the quality system of each participant (described in his own Quality Manual)
- intended degree of assurance established by the client (see clause 12.2.2.2)
- pre-existing organizations able to be integrated into the system
- more generally, technological level within the area, past practice and experience.

This variety of conditions and constraints does not make it possible at the present time to define control classes for whole building processes. Degrees of control may, however, be distinguished and even standardized (see clause 12.2.2.2).

12.2.2.2. Variation of degrees of control

Control systems can be widely differentiated. Some common ways to vary the degree of control are now given.

These various ways generally do not provide the same effects.

(*a*) More or less complete and detailed planning of the control system within the quality assurance plan. The first purpose of such planning is to avoid omissions and ambiguities. It may also specify, for example

- (i) how each control should be formalized
- (ii) control stops where the building process can be subdivided, mainly at points where responsibility is transferred, from one party to another or where one phase of the process gives way to another.

A precise formalization is highly motivating and constitutes by itself a guarantee. See the General Principles on quality assurance for structures, clause 5.2 (published in 1981 as IABSE Report No. 35).

(*b*) More or less specifying the independence of controllers.

Independence is a relative and imprecise property.

(*c*) Selecting sufficiently qualified controllers, with regard to the activity to be controlled and to the control criteria.

(*d*) Combining, if appropriate, several control methods, exerted by different bodies, for the same purpose.

A duplication of the same controls made by two different persons may be demotivating for both. Conversely a good example consists of supplementing controls of execution by controls of the completed structure, such as defined in subsection 12.2.6.

(*e*) Differentiating the intensities of controls; for example, in the case of statistical control, it may consist of differentiating the definition of batches or units, the sampling procedure, the sample size.

This differentiation directly modifies the filtering effect of control, but also indirectly influences the strategy of the producer, because he considers the costs of rejection or correction to optimize the level of quality of his supplies. This indirect effect is commonly the main one. Hence, the main effect of a control is its mere existence.

(*f*) Severity of acceptance criteria.

In the case of statistical control, this severity cannot be measured only by a level of confidence, because it depends also on the reliability (and hence on the consistency) of the prior information upon which the acceptance criteria are based.

(*g*) Severity of actions taken in case of non-compliance.

12.2.2.3. Certification

Certification may be envisaged for design, materials, components and execution.

ISO has defined several types of certification. When these types were established no particular consideration had been given to construction activities. Other definitions and terminologies are in discussion within the EC.

The various types of certification give unequal degrees of reliability. More precisely these differences result from the general possible degrees of differentiation listed in clause 12.2.2.2, and more specifically from

The permanent character of a certification is by itself favourable to its reliability.

(*a*) who certifies (the producer or another body)
(*b*) who designates a certifying body external to the producer
(*c*) who approves the quality system of the producer (the producer, the certifying body or another body such as e.g. a Commission or a public authority)

A designation by the public authorities gives generally the best independence.

(*d*) whether, how and by whom the internal control is supervised

A higher reliability is obtained if the supervising body from time to time samples certified products from the stock and tests them.

(*e*) whether sufficient sanctions can be expected if it is discovered that certain products do not comply with the certification requirements.

12.2.3. Control of planning and design

This control basically consists of

- verifying that all necessary requirements and conditions are satisfied, for the completed structure and for its execution
- checking that the calculation models and methods are appropriate and that the numerical calculations are carried out correctly
- checking that the drawings and descriptions are clearly understandable and that they comply with the design calculations and with the established specifications.

For this checking three general methods are defined in Bulletin 191. They are described and commented in detail in IABSE Surveys S 27-484.

According to the contract, the control may have

- to verify also that the required quality of planning and design is reached

This quality shall be specified where appropriate. For example various degrees of quality of designs are defined in general terms in the Proceedings of the IABSE Workshop RIGI 1983 and in CEB Bulletin 184, par. 4.1.2.

- to be exerted globally or at each of a series of predefined phases.

A control exerted at successive phases is more rigorous than a control exerted globally. In the first case the content of the individual phases should be rationally chosen (see, for example, IABSE 27-84).

12.2.4. Control of materials and structural components

12.2.4.1. General

The materials and components considered in this clause are

- all steels, anchorage devices, ducts
- primary (raw) materials for concrete, mortar, grout, i.e. cement, aggregates, water, admixtures
- concrete (industrially produced, 'ready-mix concrete', or produced on site), components if industrially produced.

Although not explicitly considered in this clause, the other structural materials to be used (e.g. bearing pads, glue, epoxy, products for curing, mortar, grout, materials for formwork and falsework) should be controlled by following the same principles. Their control should, however, be specifically differentiated.

Among these materials and components a distinction is made between those which are covered by an approved Quality System (QS), those that are not and those which are produced on site by a contractor or subcontractor.

For components produced on the site, subsection 12.2.5 is relevant.

12.2.4.2. Materials and components covered by an approved QS

The quality control made by the producer shall be supplemented by a limited series of controls intended to cover the interface with the supplier and the transferring phase.

If relevant, properties not covered by the approval shall be identified and controlled as in clause 12.2.4.3.

(*a*) For all steels the origin and identity of every delivery shall be controlled. This should be made by referring to the documents of certification carried on the delivery, to labels and to rolling marks (where relevant). The shape or straightness and the surface conditions shall also be controlled.

(*b*) For anchorage devices, cement, admixtures and aggregates the origin and identity of every delivery shall be controlled, along with

(i) for anchorage devices the surface conditions
(ii) for cement the temperature and, if relevant, the colour
(iii) for admixtures the limit date of use, if relevant
(iv) for aggregates, if not standardized, the grade (dimensions), cleanness and other measured properties mentioned on the delivery note.

(*c*) For ready-mix concrete, the following should be controlled for every delivery

(i) the origin and identity of the mix (components, notably additive, shall be defined in a delivery note established by the producer)

It is recommended to control the absence (or better the impossibility) of addition of water during transport or at delivery.

(ii) the interval of time between mixing and delivery

Controls of consistence on specimens taken from the delivery are recommended.

(iii) sometimes the temperature (especially during severe climatic conditions).

Strength tests shall be made for compliance control, as defined in subsection 12.2.4, with an appropriate frequency.

The differentiation to concrete produced on site (no approved quality system) shall be established in terms of a lower number of tests and acceptance criteria based on more favourable prior information, and not in terms of a lower γ_c-factor.

(*d*) For structural components, the following are to be controlled for every delivery

(i) the origin and identity, with special regard to the reinforcement if the factory produces components having the same external dimensions but different reinforcement; this shall be made by reference to the delivery note and to marks
(ii) the indication, on the delivery note, of all required information on the time and conditions of production
(iii) the dimensions and deformations, the absence of spalling or cracks
(iv) the colour, if relevant.

The quality system rules applicable to ready-mix concrete are widely applicable for the production of industrialized structural components.

12.2.4.3. Materials and components not covered by an approved QS

A detailed programme of control shall be defined.

For materials to be controlled statistically, some controls at least shall be made at the factory or the production site, in order to ensure that the production is not excessively heterogeneous rendering the programme of control not significant.

In the absence of any reliable prior information, no statistical control can be simultaneously operational and reliable. A total control is possible only if non-destructive test methods are available and practical.

12.2.4.4. Structural concrete produced on the site

For structural concretes, the properties likely to be specified are various. The most common are the size (or maximum size) of aggregates, the consistence and the required characteristic compressive strength $f_{ck\ req}$. For lightweight and heavyweight concretes, the unit mass is generally specified.

There are also various ways to specify these properties. Hence, the corresponding quality controls can be very varied.

For non-structural concretes, which commonly are a non-negligible part of the total production, controls should be alleviated.

For example concretes may be specified as designed mixes or prescribed mixes (see for the definition Appendix d, section d.3).

Unless a low degree of control is accepted, the production control should include

- the production equipment (weighing and gauging equipment, mixer and control apparatus)
- the production process
- some concrete properties, first as preliminary tests and then during production.

For compliance control the total production should be divided into lots which are considered to be produced under the same essential conditions and can be judged separately.

For each lot several batches are sampled, and from each batch one or several specimens are made for tests.

The specimens are made, preserved and tested according to standardized conditions.

For many properties, e.g. consistence, the conformity of the lot under consideration is recognized only if all individual test results comply with the specified values of these properties.

Irrespective of the number of specimens taken from each batch, the test result x_i to be used in the compliance strength criterion will be only one for each batch, equal to the mean value if more than one specimen is taken from the batch.

For concrete strength, the compliance control generally shall be statistical, and the criterion is defined by the two following inequalities

$$\bar{x} \geqslant f_{k\,req} + k_1$$

$$x_{\min} \geqslant f_{k\,req} - k_2$$

where

$\bar{x}$ is the mean value of the n test results $x_1, x_2 \ldots x_n$
$x_{\min}$ is the minimum value of these test results
k_1, k_2 are specified coefficients; for $n \geqslant 6$, k_1 is taken equal to λs_n where

$$s_n = \sqrt{\left[\frac{\sum_i (x_i - \bar{x})^2}{n-1}\right]}$$

k_1 (or λ) and k_2 may be dependent on n. Their values should be deter-

In the case of production certified in certain precise conditions, tests made on concrete properties may be used for compliance control (see section 12.1).

For concrete not produced on the site, production control may have to cover transport to the site, depending on the delivery conditions.

As measures to be taken may be different for various structural elements it is recommended to avoid that a lot includes elements of different kinds such as beams and columns. For practical reasons, it is recommended to define the lots such that the same workmanship and climatic conditions can be presumed for each lot.

The term 'same conditions' implies that any important variation in the quality of the constitutive materials (for example due to different deliveries, or different climatic conditions) should be excluded.

The first inequality generally is the main one. The second one is useful to check whether an important anomaly has occurred during the production of the lot, and may be dominating for small n.

No final generally agreed decision has been taken on the limits of n; $n \geqslant 6$ is taken from ENV 206; greater values such as $n = 15$ in MC 78 are hardly compatible with any practical division in lots.

The distribution within a lot may generally be considered to be Gaussian. The prior information on the parameters of this distribution may generally be put together in a hierarchical model (see JCSS reports) which is based

on practical experience and is different for different standardized conditions, relating to production and production control.

After the standardized conditions have been defined and the associated prior information has been numerically assessed, combinations of this information with k_1 (or λ) and k_2 values make it possible to draw 'OC lines' giving the probability of acceptance of the lot as a function of the actual ratio of defectives.

For the same reasons the intensity of these controls can only be differentiated

- either qualitatively (defining those tasks that will be controlled), or
- indirectly by the means devoted to inspection (e.g. permanent presence of a surveyor from the controlling body).

All differentiations in quality control shall be defined within the frame of the quality assurance measures. Although widely recommended, establishing a more or less detailed site journal cannot be considered as an alternative to control reports. It gives only an indirect differentiation.

For production refer to subsection 12.2.4.

Other activities to be controlled, if relevant, are protection with regard to frost and steam curing.

mined on the basis of statistical studies taking into account prior information on the type of distribution, on the standard deviation and on the mean value. This information is in every case associated with the observance of standardized conditions.

12.2.5. Control of execution

12.2.5.1. General

The following shall be controlled

- the interfaces with design and supplies of materials and components
- the workmanship.

These activities are not able to be fully standardized. The corresponding controls are either purely qualitative, or, if there are quantitative references (e.g. geometrical tolerances) cannot be based on statistics, so that their conclusions generally are qualitative.

Each individual control activity shall be documented by a record, a note or a report.

For concrete structures the main activities to be controlled relate to

- concrete
- formwork and falsework
- reinforcement
- prestressing steel and devices
- precast units.

12.2.5.2. Concrete

The main activities to be controlled are

- transport and placing (limit duration and avoid segregation)
- compacting
- curing (ensure a sufficient duration)
- surface finishing.

12.2.5.3. Formwork and falsework

If the materials to be used for these are new, they should be controlled according to the principles defined in subsection 12.2.4. If not, the verification, selection and restoring of the delivered materials are activities to be controlled.

The other main activities or qualities to be controlled are

- erection (geometrical and mechanical aspects)
- tightness
- internal surface
- removal.

12.2.5.4. Reinforcement

The main activities to be controlled are

- handling and storage
- cutting and shaping (if not made in a factory before delivery)
- assembly (including laps, joints and welding if relevant)
- positioning.

12.2.5.5. Prestressing steels and devices

The main activities to be controlled are

- handling and storage
- cutting and shaping, if relevant, of ducts and tendons
- positioning
- tensioning (a detailed record shall in any case be established)
- grouting (for the materials to be used and the production of the grout, the control shall be made by following the principles defined in subsection 12.2.4).

12.2.5.6. Precast units

The main activities to be controlled are

- handling and storage
- transport on site
- positioning
- assembly.

12.2.6. Control of the completed structure

12.2.6.1. Control for final acceptance of the structure

This control consists of

- a general supervision of the structure and of the control documents produced during execution

This includes mainly the control of measures taken in case of non-compliance.

- if required by the contract, controls of some performances of the structure

They are for example control of serviceability performance by loading tests, control of watertightness, control of aesthetic required qualities, etc.

In order not to damage the structure or reduce its life duration, controls by loading tests should be limited, commonly below serviceability limit states.

- unless a low degree of control during the use of the structure is accepted, collecting and ordering all existing documentation likely to be used during the lifetime of the structure for use, maintenance and repair.

This documentation shall include, among others, as-built drawings.

12.2.6.2. Organization of later controls

For these controls the following should be established

- a utilization plan for the structure
- a programme for inspection and maintenance, to be carried out where long-term compliance with the basic assumptions for the project is not ensured.

The utilization plan should be established during the design phase, and the programme for inspection and maintenance may be defined by a pre-existing standard. These documents may, however, have to be revised and updated on the basis of the documentation mentioned in clause 12.2.6.1.

13. MAINTENANCE

13.1. GENERAL

Design should be based on available information regarding the foreseen inspection and maintenance policy, so that relevant decisions may be taken on various design parameters; such as construction joints, form and section of less accessible elements of the structure, type and quality of basic building materials, detailing, sensitivity of the structural concept etc.

Due to the character of the subject, this chapter is not meant to be operational.

13.2. INSPECTION

Structures designed and constructed in conformity with the provisions of this Code should be inspected and maintained as frequently and carefully as possible, so that they will continuously fulfil all requirements related to the intended service and safety.

Particularly, structures of major importance or under adverse service conditions should necessarily be inspected periodically, adopting appropriate in-situ testing and monitoring strategies.

National authorities should decide on inspection and maintenance policies to be imposed on concrete structures under well defined conditions.

For conventional concrete structures under normal service conditions, the following time periods between successive competent inspections could be suggested

for houses, offices, etc.	10 years
for industrial buildings	5–10 years
for highway bridges	4 years
for railway bridges	2 years
for road bridges	6 years

13.3. REPAIR

All minor defects or light damage (non-structural) impairing the performance of elements or parts of the structure should be systematically rehabilitated.

If serious damage is observed or major defects are suspected (with possible structural consequences) an appropriate assessment and redesign procedure should be followed.

Relevant information is included in CEB Bulletin 162.

PART IV. DESIGN FOR PARTICULAR TECHNOLOGIES

14. PRECAST CONCRETE ELEMENTS AND STRUCTURES

14.1. DESIGN BASIS

14.1.1. General

This chapter deals with the design and detailing considerations special to structures made partly or entirely of precast elements, with basic reference to ordinary buildings, structured in wall systems, beam-column systems, or dual systems.

The term 'precast element' refers to any structural concrete element manufactured in purpose built technical facilities which are protected from adverse weather conditions by appropriate means. Composite elements are precast in part, then completed with cast in situ concrete.

Structures are wholly precast, when made of precast elements, or partly precast, when some elements are cast in situ. Both are characterized by the presence of joints, providing mutual connection between elements in the overall structural behaviour.

In general the rules regarding resistance and serviceability requirements of design for in situ cast concrete structures apply also to precast and composite constructions, unless modified or supplemented in this chapter.

Precasting of structural elements derives from a philosophy of industrialization of production processes, aimed at improving economy and quality of buildings.

Whereas the precast units are produced at a high technological level in plants, the connections will be executed on the site, possibly in simple and quick operations.

Production and assembly needs give rise to various solutions for connecting structural parts, differing from those of 'monolithic' cast in situ structures. Thus fulfilment of the common performance requirements may be obtained by ad hoc design criteria, including structural modelling, use of materials and details, which may differ from the traditional ones.

The great variety of problems related with production and assembly of precast structures is not dealt with extensively in this Code, being a matter of specific recommendations. However, this chapter points out some criteria consistently with the principles of the Code, which apply to general aspects of precast elements and structural systems pertaining to ordinary buildings. Durability considerations are also included.

14.1.2. Structural arrangement

The layout of the structure and the interaction between the structural members should ensure a robust and stable behaviour.

Deformaton due to differential loading, shrinkage, creep, thermal effects and possible differential settlements of foundations should be accounted for.

Floors and walls composed of adequately tied precast elements may be used as wind-bracing horizontal and vertical diaphragms, provided that the corresponding in-plane forces can be appropriately transferred to the supporting structural elements.

Joints in precast structures may provide 'monolithic' connection by in situ casting of concrete over reinforcement protruding from units; thus realizing full continuity in a very similar way to cast in situ structures. The same may be obtained by post-tensioning steel reinforcement across grouted, glued or even dry joints with little interspace, or by means of only metal devices, although this might affect local stiffness and ductility. Structures with joints as above may behave as continuous frames.

Other joints may realize hinge-like restraints, able to transmit mainly shear and compression. In structures connected with such joints, bracing is needed.

The necessary interaction between elements is obtained by tying the structure together, using acting horizontal (peripheral and internal) and vertical ties.

Structural integrity (with particular regard to progressive collapse) shall be provided by an adequate 'strategy', choosing active and passive measures for controlling the occurrence and the propagation of damage. Engineering judgement is required in assessing these strategies for particular structures.

Elements themselves may be partly precast and completed in situ (composite elements) with large surfaces as interface connections, with or without transverse reinforcement, to give monolithic behaviour.

In general, precast elements have limited shrinkage and creep deformations after assembly. However, effects of differential values, particularly in composite members, should be evaluated. This may affect, among others, the spacing of expansion joints.

14.1.3. Analysis and design

The analysis of a precast structure should be based on assumptions compatible with the structural layout and with the actual detailing of the structure. Possible changes of the statical system during the different stages of construction should be assessed.

In addition to the limit-state requirements, the design (especially of connections) should account for an easy and reliable assembly and maintenance.

Joints should be adjustable during assembly and—especially those made of materials other than concrete—be protected from aggressive agents. They should furthermore, if necessary and possible, be inspectable and replaceable.

The analysis should account for

- the behaviour of structural units as such, in subsequent functional phases (including transitory situations: see subsection 14.1.4)
- the behaviour of structural assemblies, in particular with regard to actual deformability, strength, and fatigue resistance
- the uncertainties influencing restraints and force transmission between elements, that may depend on errors in geometry and in the positioning of units and bearings
- all external and internal actions intervening in all phases; some non-conventional conditions may be envisaged (e.g. for dimensioning against overall instability, explosions, impacts, progressive collapse).

When the production is industrialized and continuously monitored and a complete quality assurance system is supervised and certified by an independent body, which implies systematic rejections in case of non-compliance, the partial safety factors γ_c and γ_s may be chosen between 1.5 and 1.4 and between 1.15 and 1.10, respectively, depending on the reduction (up to 50%) of the tolerances defined in subsection 1.4.5.

No reduction of γ_c is justified by the mere use of prefabrication, nor by a reduction of the standard deviation of concrete strength, γ_c being intended to be applied to an unchanged specified 5% fractile. A smaller standard deviation implies a smaller mean strength, and the fractile 5% is such that the safety degree is not significantly modified. However, a known smaller standard deviation may be taken into account in the acceptance criteria of concrete.

Usually the quality assurance system can ensure better compliance with the specified tolerances on formwork and reinforcement for precast than for cast in situ elements.

The reduction of γ_c is applicable only to the precast elements, and not to the joints, for which section 6.10 is applicable.

Under the same conditions, in case of continuous production of identical elements and making a direct statistical assessment of the performance of the whole production possible,

- either another reduction of γ_c may be made by dividing it by 1.1η if the mean ratio η of the strength of drilled cores and standard specimens is greater than 0.9

This other reduction of γ_c implies that it has been experimentally and statistically demonstrated that the quality of workmanship justifies a reduction of the mean conversion factor η, normally taken equal to 1.1, included in γ_c. In the direct assessment of R_d a possible reduction of the dispersion of the conversion factor is implicitly taken into account.

The two possible reductions of γ_c may be accumulated.

- or the design resistance R_d may be directly assessed according to the rules relating to statistical interpretation in the case of design by testing.

All reductions of partial factors envisaged in this subsection may be taken into account in the design on the basis of prior information if statistically demonstrated.

14.1.4. Transitory situations

Precast units should be designed to make all operations of demoulding, handling, storage, transport and erection safe and to avoid undesirable effects on their future behaviour within the structure.

All static and dynamic transitory situations the units may undergo should be verified in the design, accounting for actual material properties at their time of occurrence. These may be determined by means of reduced sampling and simplified criteria. Partial safety factors may be reduced for verifications in transitory situations during handling and transportation.

Lifting devices and loops must have ductile behaviour and be dimensioned accounting for possible uneven lifting, for dynamic actions, and for lower bond capacity in the early stages.

The verification of safety of the structure during assembly should be performed for a combination of actions, such as wind, dead weight and possible superimposed load, with their respective eccentricities, expected to act during assembly.

The edges of elements containing supports for security devices (such as railings and lifelines) should be checked for a nominal load of 2 kN in the most unfavourable position. Floor elements should allow for passage of workers, and be able to carry a nominal vertical load of 1.4 kN in dwellings and 2 kN on any 200 × 200 mm accessible square.

The sampling of concrete at the time of particular operations, like prestressing or demoulding, may be less extended than normally required. Thus, conventional characteristic values may be worked out on the basis of empirical formulae.

Reduced safety factors may be justified by the limited probability of actions exceeding the nominal values and the limited consequences of failure before normal use (see section 2.2 of CEB Bulletin d'Information No. 191 'General Principles on Reliability for Structures'). On the other hand, effects on the normal safety requirements must be considered.

Possible reduction of steel ductility at low temperatures should be taken into account.

Unless more accurately determined, dynamic vertical effects of lifting and transport may be accounted for by treating the self-weight with a factor 1 ± 0.2.

Overloads due to demoulding might be measured.

Dynamic horizontal effects of transport should be prevented by bracing devices.

14.1.5. Tolerances

Allowance for construction inaccuracies should cover deviations in the production and the erection of the elements. They may be assessed from a statistical analysis of measured or predicted deviations.

It should be checked that the tolerances in the actual execution are compatible with the tolerances assumed in the analysis and design.

Tolerances affecting structural behaviour should be pointed out in the design documents.

For tolerance limits, refer to subsection 1.4.5.

Production of prefabricated units may be more accurate and may be subject to stricter quality control than in situ cast structures.

14.2. ELEMENTS

14.2.1. General design considerations

Dimensioning of concrete elements is performed in general according to the methods of dimensioning and detailing of concrete members presented in this Code.

Where such methods are not applicable, particular models may be used. Performance should be verified by means of behaviour models worked out on the basis of experimental tests, or by referring to acknowledged technical recommendations and technical approvals. Testing procedures and results should be adequately documented.

Production and assembly considerations may lead to innovative arrangements, in which sufficient safety and serviceability is obtained in a manner different from those normally utilized in cast in situ structures. Such deviations from traditional dimensioning and detailing rules require specific justification.

Especially connections may require global behaviour models (e.g. moment rotation, shear-displacement) rather than models on stress–strain relationships. Justification by testing may often be needed. Uncertainties of behaviour should be accounted for by adequate idealizations and safety factors.

In the design stage, appropriate action should be taken to facilitate correct execution.

Good design should minimize likelihood of errors on site, foresee assembling operations apt for visual control, and prescribe realistic tolerances. Further requirement may be a proper demountability of the structure.

14.2.2. Execution

(*a*) *Manufacture*

Installations, raw materials, working processes, etc. should satisfy the assumptions used in the design.

In order to avoid harmful chemical reactions during the manufacture of units, the temperature of concrete should not exceed specific limits appropriate to the concrete mix used. Different properties of concrete in different zones of the element, according to the type of casting should not impair structural behaviour.

(*b*) *Handling, storage and transportation*

Each structural element should have an identification mark.

Identification marks should give all necessary information with respect to identification and erection.

Handling, storage and transportation of the elements should conform to the design specifications.

Care should be taken to avoid differential deformations during storage.

(*c*) *Erection and assembly*

During erection, adequate safety against instability should be provided.

Erection of elements should be in accordance with design assumptions; e.g. minimum support length, contact stresses, bearing material, etc.

Components should not be installed if they have been damaged in a way which affects their structural performance.

Considerably damaged components should be reassessed for use in normal or reduced performance, repair, or rejection.

Connections should satisfy the design functions with regard to safety and serviceability (including durability considerations).

14.2.3. Reinforcement detailing

(a) Concrete cover

Nominal values of concrete cover may be reduced by 5 mm with respect to the values specified in Table 8.4.1, if accurate control on bar position and on concrete compaction is performed.

Reduction of concrete cover is related to better compaction in precast units than those specified in Table 8.4.1. Where fire resistance is needed, other minima may apply.

(b) Detailing of support areas

Detailing of bars near the bearings of precast elements shall be carefully done to ensure that the shapes prescribed can be placed in the positions required by the design.

14.2.4. Composite elements

The design of a composite element should be based on the material properties of the combined materials (strength, shrinkage, creep, thermal deformations).

Composite members may be considered monolithic, with respect to their resistance, when shear transfer can be performed safely (without discontinuous deformations at the interface), by pure interface bond or by interaction with connecting reinforcement.

The effectiveness of connecting reinforcement and its contribution to the failure mechanism in ultimate limit states shall be verified.

A composite element is a structural member composed of a precast part of reinforced or prestressed concrete, and a cast in situ part, connected to the precast part by bond, with or without reinforcement connectors.

The properties which affect shear transfer at the interface surface are: surface roughness and cleanness, concrete strength, strength of connectors, deformability and bond properties. The shear reinforcement should be anchored properly on each side of the interface. The splitting force caused by the shear reinforcement has to be considered, when significant. For composite floor members with flat and wide interfaces subjected to low shear stresses, it is allowed to rely upon friction at the interface only. Connecting devices across it may be omitted, see clause 14.4.2.2. However, the interface should be rough and properly cleaned before cast in situ concreting, the top layer adequately compacted and cured in order to prevent high early shrinkage.

For design in low shear, reference is made to FIP Guide to good practice on 'Shear at the interface of precast and in situ concrete'.

14.2.5. Construction details

14.2.5.1. Bearings

The integrity of bearings for precast members should be ensured by the effectiveness of reinforcement on both sides of the reinforced bearings and by a suitable limitation of the bearing stress.

Account should be taken of possible horizontal or flexural actions at the bearing due to creep, shrinkage and temperature effects, unforeseen eccentricities, etc., by either the provision of sliding and rotation bearings or suitable reinforcement.

In order to satisfy the requirements of stability and resistance, the support arrangements should consider

- limitation of the contact stresses (a_1)
- additional support lengths taking into account concrete cover on the main reinforcement of both supporting and supported element ($a_2 + a_3$).

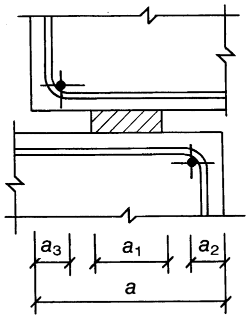


Fig. 14.2.1. Support length

When horizontal reactions cause friction movements, the possible accumulation of non-reversible shifts due to uneven behaviour under cyclic actions (e.g. thermal) should be prevented.

Support zones should be dimensioned and detailed to assure correct positioning and resistance, accounting for production and assembly tolerances.

Possible local effects of prestressing anchorages should be considered.

The support length should normally satisfy the condition (net of tolerances)

$$a \geqslant a_1 + a_2 + a_3$$

Anchorage length of reinforcement should also be accounted for.

For masonry or plain concrete supports, it may be assumed that $a_2 = 25$ mm.

The shape of the sections of reinforced and prestressed concrete floors should be adequate to fulfil the requirements related to strength and serviceability (including durability considerations), as well as fire resistance.

14.2.5.2. Floor elements

Prestressed floor units should be designed and detailed with due regard to the risk of splitting and bursting in the anchorage zones of the prestressing tendons.

The overall depth and the thickness of flanges should comply with punching resistance, transverse distribution of loadings and transverse deflection limits.

14.2.5.3. Beams

The cross-section thickness of beams should not be less than 50 mm in their final presentation.

Simply supported beams should have sufficient torsional restraint to withstand horizontal actions due to wind, unsymmetrical loadings, imperfections and impacts.

The support, both on beam and on column or wall side, should be designed for an unintentional additional torsional moment

$$T_{ad} < V_{Sd}l/300$$

where

l is the beam span
V_{Sd} is the design vertical reaction (shear force)

and for a horizontal force

$$H_{sd} > 0.2V_{Sd} > 30\,\text{kN}$$

Lateral instability of the beam due to bending and torsion should be studied accounting for an unintentional horizontal misalignment at midspan

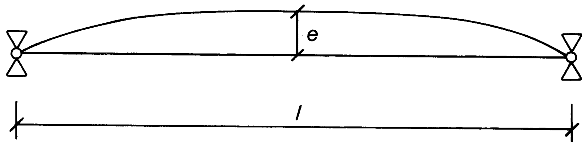


Fig. 14.2.2. Horizontal eccentricity e *at midspan*

$$e > l/500$$

in addition to external actions.

Accurate account must be taken of realistic unfavourable restraint conditions.

14.2.5.4. Columns

(a) Longitudinal reinforcement
Pretensioned reinforcement may be accounted for in computing minimum reinforcement (see clause 9.2.3.1); minimum bar diameter does not apply.

(b) Reinforcement details
The provision of reinforcement against possible splitting at column ends should be considered.

(c) Metal footings
Column metal bases should be designed for both erection loads and loads which occur in service.

Erection loads are often more critical because before grout is placed under the metal footing, all the loading is taken via the anchor bolts and/or shims.

The column base should be anchored to the main column reinforcement.

The necessary anchorage depends on the design forces on the column base only.

(d) Corbels
Corbels should be capable of resisting vertical and horizontal actions due to loading, creep, shrinkage, thermal movements, eccentricities due to tolerances etc. (see also clause 14.2.5.1).

The main tensile reinforcement of concrete corbels should be anchored positively near the extreme outer face, e.g. by welding cross bars or by welding to confinement angles. The eccentricity of the loading should be kept to a minimum.

In case of embedded structural steel corbels, the resistance of the concrete against crushing should be verified.

14.2.5.5. Foundation elements

Concrete sockets should be able to transfer vertical loads, bending moments and horizontal shear from columns to the soil. The joint between the column and the socket should be large enough to enable a good concrete filling below and around the column.

The joint-filling under the column base should enable good force transfer between the column and the foundations.

Vertical force is assumed to be transmitted entirely through the column base unless the lateral surfaces of column and socket are provided with adequate roughness or keys.

Bending and shear are assumed to be transmitted entirely to the socket walls.

14.2.5.6. Wall panels

(a) Plain concrete panels

Minimum concrete grade of panels should be C16.

Concrete panels should be designed as plain elements if the vertical reinforcement is less than $\rho = 0.003$. Panels shall contain boundary reinforcement to control shrinkage and thermal cracking.

Reinforcement to control cracking caused by in-plane shear shall be placed in the centre plane, or symmetrically.

The precast elements may contain also structural continuity reinforcement to be joined in situ with reinforcement of adjacent elements in order to produce a wall system, as required in section 14.5.

Local reinforcement of wall panels comprises the lifting devices and reinforcement to prevent structural damage during handling and transportation.

The thickness of panels for bearing walls should normally not be smaller than 100 mm, if made of normal concrete and not smaller than 140 mm, if made of lightweight concrete. Shear walls and flank walls made of normal concrete may not be less than 80 mm thick.

The above values should be increased by 20 mm for external walls.

It is recommended that connections for lifting panels from the top edge should be anchored to a depth taking into account dynamic and demoulding overloads, as well as concrete strength at demoulding time. Very severely loaded devices are to be anchored according to structural analysis. Unless analysed, the depth can be taken as 0.75 of the panel height.

Lifting points in the base of the panels should have adequate bearing capacity.

(b) Reinforced panels

Minimum concrete grade of reinforced panels should be C20. Wall panels may be considered in the structural analysis as reinforced elements if the reinforcement ratio is $\rho \geqslant 0.003$ in both directions.

The main reinforcement should consist of two meshes, placed near the external surfaces of the panel when its thickness is $\geqslant 150$ mm.

(c) Sandwich panels

Sandwich panels designed as composite elements should be verified by analysis or by testing.

The connectors between the external leaf and the bearing leaf should be made of corrosion resistant material. The design should account for possible different kinds of physical and chemical aggression, and for fatigue actions.

A sandwich panel normally consists of an internal bearing leaf, designed in accordance with the previous structural requirements (see (a) and (b)), and of a thermal insulation layer and of an external leaf. The latter should be free to deform independently; otherwise the curvature due to temperature and shrinkage shall be accounted for.

Care should be taken in controlling mesh position and concrete quality of the external leaf with respect to durability problems.

14.3. JOINTS

14.3.1. General

Joints must be designed to transmit all action effects implicit in the assumptions made in the analysis of the structure as a whole and in the design of the individual members to be joined. The design should ensure that

Particular attention should be paid to the detailing of the joint, in order to prevent premature splitting of spalling of concrete at the ends of precast units. Tolerances and fitting requirements, as well as construction requirements with respect to convenient completion and work inspection, are to be taken into account when dimensioning the joint.

- the joint is able to accommodate the relative displacements needed to mobilize the resistance of the joint
- the joint is able to resist all action effects resulting from the analysis of the structure as a whole, as well as those resulting from the analysis of the individual members
- the strength and deformability of the joints secure a robust and stable behaviour of the structure as a whole
- account is taken of the tolerances anticipated in manufacture and erection.

When verifying the ultimate limit state of the joint, the resistance of the in situ portion, as well as the adjacent portions of the precast elements, is to be considered.

The resistance and the stiffness of the joints may be based on analytical methods, or on laboratory tests. The influence of workmanship imperfections at the site is to be taken into account. Unfavourable deviations from testing conditions should be evaluated when using 'design by testing'.

Compression, flexural and shear joints are defined by the primary action effect they sustain.

Tests may be especially useful to determine the sensitivity of strength and stiffness parameters to the expected variations in geometry and in workmanship.

The design should allow for effective assembly and filling operations; solutions that are highly sensitive to variations that cannot be controlled and which might impair the durability should be avoided.

14.3.2. Compression joints

This term is used for joints designed to transmit axial, or slightly eccentric compressive forces between precast units (such as columns, wall panels or struts) and other building elements.

The strength of mortar joint material should not be less than 70% of the adjacent precast concrete strength, if the mortar is not transversely confined, unless the joint is designed accounting for actual mortar strength.

Dry joints with no intermediate padding material may only be used where great accuracy in manufacture and installation is obtained.

Bedded joints with mortar, concrete or hardening polymers as padding material may be used, provided all necessary precautions are taken to prevent relative movement of the connected surfaces during hardening of the padding material.

Relatively weak joint material can initiate cracking in precast element, parallel to the direction of loading, and reduce the capacity of the joint.

The shape of the joint may affect its resistance.

The minimum and maximum dimensions, for joints filled after the unmatched elements are positioned, are chosen taking into consideration proper filling and compaction.

Account should be taken of unintentional eccentricities.

In assessing the resistance of compressive connections, the interaction between axial and shear forces should be accounted for.

Additional care is needed for appropriate detailing of longitudinal bars across the joint.

(*a*) *Joints between plain concrete elements*

The resistance depends upon the eccentricity of the normal load, the strength and confinement of the in situ concrete and/or of the mortar, and the fixing moment from the floor. This applies also to hinged joints between reinforced elements. If $V_{Sd} > 0.1 A_j f_{cd}$ the reduction of N_{Rd} should be evaluated by means of interaction diagrams (see Bulletin No. 169).

Interaction of N and V may be disregarded if

$$V_{Sd} \leqslant 0.1 A_j f_{cd}$$

(*b*) *Continuity joints*

Continuity joints between reinforced elements are the most frequent type of horizontal joints of columns in precast structures.

In this kind of joint, the vertical compression force N is transmitted through the direct contact of the connected elements and/or through the connection of reinforcement.

14.3.3. Shear joints

Shear joints appear in wall systems, in bracing walls interacting with beam and column systems, in prestressed segmental members, in floors for horizontal diaphragm action, as joints between precast and cast in situ concrete in composite members, and as beam to beam, column to column or beam to column joints in systems of prefabricated linear elements.

Plane and keyed joints are distinguished.

The structural requirements for shear joints refer also to flexural joints acting as shear joints as well.

Shear joints with negligible normal force are recommended to be keyed joints.

14.3.3.1. Ultimate resistance

For verification of the ultimate limit state of a shear joint it may approximately be assumed that the stress distribution is constant along a definite section of the joint length.

The design resistance V_{Rd} of joints which do not allow inspection for proper filling with concrete, is to be reduced by applying a supplementary $\gamma_{Rd} > 1.0$.

Possible interaction with tensile forces should be accounted for.

The design resistance of the shear joint may be calculated, making use of the formula

For γ_{Rd} the following values may be used

$\gamma_{Rd} = 1.3$ for open keyed joints or composite elements
$\gamma_{Rd} = 1.5$ for closed keyed joints
$\gamma_{Rd} = 1.6$ in the general case of plain joints.

$$V_{Rd} = \min \begin{Bmatrix} \{\mu[A_s f_{yd}(1 + \cot\alpha)\sin\alpha + N_d] + 0.1 A_k f_{cd}\}/\gamma_{Rd} \\ 0.3 A_j f_{cd} \end{Bmatrix} \qquad (14.3\text{–}1)$$

where

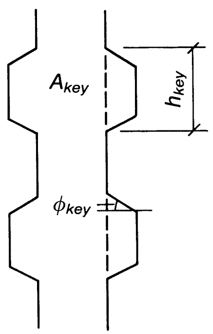
μ is 0.5 for smooth plane surfaces and 0.9 for rough or keyed surfaces
A_s, f_{yd}, α are the cross-sectional area, design yield stress and inclination of the steel bars crossing the joint and being well anchored on both sides
N_d is the design (most unfavourable) normal force acting on the joint section (positive when compressive)
A_j is the cross-sectional area of the joint under compression
A_k, f_{cd} are the cross-sectional area of the portion of the joint keys interacting in the resistance, and the design strength of concrete in the considered joint portion ($A_k = 0$ for plain joints or when $A_k/A_j < 0.2$).

An appropriate value of minimum transverse reinforcement is needed in order to secure a minimum ductility and to counterbalance uncertainties related to the determination of action effects and to the accuracy of models.

The simplified verification formula is based partly on a shear-friction mechanism, it accounts for the additional contributions of possible keys, and it is derived from series of tests on overall joint samples.

In case of tensile forces acting on the joint, N_d in eq. (14.3–1) is given a negative sign. When compressive, the most unfavourable N_d should be used.

More sophisticated models for specific cases including also dowel effect, may be found in chapter 3.



$A_j = b_j l_j$

$A_{key} = b_{key} h_{key} n$

l_j – joint length (e.g. storey height)

n – number of keys at length l_j

$h_{key} \geqslant 50$ mm and $t_{key} \geqslant 10$ mm

$h_{key}/t_{key} < 8$

$\phi_{key} \leqslant 30°$

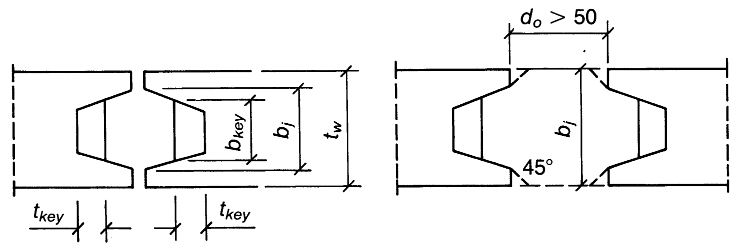


Fig. 14.3.1. Definitions—keyed joints

A minimum reinforcement ratio is advised (see Bulletin No. 169)

$$\frac{A_s f_{yd}}{A_j f_{cd}} \geqslant 0.01$$

14.3.3.2. Stiffness

The determination of load effects may assume for simplification elastic behaviour of the structures. Individual parts of the structures, however, may be assigned a stiffness corresponding to the actual load level.

In the absence of more detailed information, the elastic shear stiffness C_j of the joint, applicable for linear analysis of a precast structure under normal actions, may be assumed as

$$C_j = 5 \times 10^3 \, V_{Rd}/A_j \text{ (MPa/m)}$$

where V_{Rd}, A_j are the design resistance and cross-section area of the shear joint.

14.3.4. Flexural and tensile joints

The ultimate limit state is to be verified neglecting the tensile strength of the concrete.

Besides the resistance to bending moment and/or to axial force, the resistance of the joint to shear is to be verified. The requirements given in subsection 14.3.3 then apply.

Continuity of reinforcement through the joint shall be secured. The jointing method used should be such that the assumptions made in the analysis of the structure are realized.

The reinforcement should be anchored in such a way that its full capacity can be mobilized.

Care should be taken to avoid bond failures of the bars also under accidental loadings.

Any type of connection that may not be justified on the basis of provisions of this Model Code shall be checked by testing.

Connections transmitting bending moments between floor slabs and precast walls, as well as beams and columns are categorized as flexural joints.

The bending moment acting in the joint depends on the stiffness of the elements and on the stiffness of the joint itself.

Continuity may be obtained by

- lapping of bars
- welding of bars
- reinforcement grouted into apertures
- overlapping reinforcement loops
- sleeving
- threaded couplers

or any other type of connection that can be shown capable of performing adequately.

14.3.5. Ties

The diaphragm action of floor and wall as required in subsection 14.1.2 should be secured by ties (in two orthogonal directions) effectively continuous and anchored at the perimeter of the structure.

Ties may be provided entirely within in situ concrete toppings or connections, partly within in situ concrete and partly within precast members or entirely within precast members.

Lapping to provide continuity of bars used for ties should be avoided if possible. Where lapping is used, the lap should be established as far away as possible from critical joints between elements and should be spiral—confined within the bond length.

Ties are reinforcements (concentrated or not) running in walls or floors from end to end, to realize the tensile bars of ideal horizontal and vertical truss systems, providing for normal and accidental situations.

Ties may be post-tensioned.

Connecting all the bars in one zone (plane) of the joint is permitted. The influence of such an arrangement on the structural reliability should be appropriately taken into account, see also subsections 14.4.1 and 14.5.1.

14.4. FLOOR SYSTEMS

14.4.1. General

The main structural requirements of floors are span load bearing, transverse load distribution, diaphragm distribution of horizontal actions, as well as resistance against accidental actions affecting the floor or its supporting structures.

Most common precast floor types are

- hollow-core reinforced or prestressed slabs, with or without in situ topping
- composite floors made of precast plates, completed with in situ solid or hollowed concrete grouting
- solid slabs
- ribs with in situ topping grouted on thin transverse slabs or on hollow infilling blocks
- double tees, with or without topping.

Floor ties are dimensioned according to the structural analysis, particularly with regard to diaphragm action.

Some specific design criteria (when needed) rely partly on special primary resisting schemes, justified by redundancy of the structure and by the provision of subsidiary resisting schemes, mobilizing the resistance of ties.

However, appropriate minimum ties should be provided in order to counterbalance uncertainties related to the determination of action effects and to the accuracy of models.

If not otherwise calculated, the following minimum design ultimate forces should be provided

- peripheral ties: 60 kN,
- internal ties: 20 kN per metre width.

14.4.1.1. Common design criteria

(a) Support arrangements

The precast floor units may be simply or continuously supported. The connections at the supports should be designed and detailed accordingly. Unintentional restraint effects at the support of simply supported floors should be appropriately considered.

The top reinforcement of continuous or cantilevering precast floors can be anchored in the precast elements or in a structural topping layer. In the ultimate limit state, transfer of stresses between the top layer and the precast units should be appropriately assured.

For floor elements manufactured by long-line methods, the way of cutting the elements may affect the support area and should be considered when the normal support length is prescribed.

(b) Longitudinal joints

When transverse load distribution or diaphragm action is assumed, longitudinal joints between precast floor elements should be designed to transfer the vertical and longitudinal shear forces from one element to the other.

The shear forces can be transferred along joint interfaces (by means of concrete or mortar), or by concentrated shear connectors, or in cast in situ toppings.

The transfer of vertical and horizontal shear forces can be realized in different ways

- grouted joints
- welded connections
- reinforced topping on the floor elements.

Possible combinations of horizontal and vertical shear should be considered in the design of the connections.

The joint faces of the precast members should be suitably rough or indented in order to make shear transfer possible by friction or interlocking effects.

Minimum joint width and minimum joint opening in the top of the floor should be designed considering the actual type of grout and grouting method. If tie bars are placed and anchored in the joint, the joint width at the tie bar level should not be less than twice the bar diameter, or 25 mm.

The strength of the joint fill, the quality of the grout, the size of the joint and the joint filling operation may affect the shear capacity and should be considered in the design and detailing.

(c) Transverse load distribution

Transverse load distribution between adjacent floor elements under action of live loads across the joints should be ensured by appropriate shear transferring connections. The shear capacity should be determined.

When the connections at the joints are not designed for bending moment capacity and provided with reinforcement or metal devices accordingly, they may be assumed to act as shear transfer hinges.

(d) Diaphragm action

Precast floors can act as diaphragms for transferring horizontal forces to the bracing vertical elements when the following conditions are satisfied

- the completed floor is analysed under realistic assumptions of the deformability of the bracing members, the precast elements and the connections
- the elements are connected and the completed floor is provided with a tying system so that lateral force transfer is possible by arch or truss action
- the tying system is able to resist all tensile forces deriving from the in-plane actions (bending, shear, tension)
- force transfer capacity is provided to the bracing vertical elements in order to realize the necessary interaction; possible stress concentrations at the joints should be considered in detailing
- in shear joints formed by longitudinal grouted joints, the average shear stress should not exceed 0.10 MPa in the ultimate limit state, if the joint faces are not provided with vertical indentations; if indented or keyed joint faces are used, higher values may be adopted, based on adequate experimental data.

In most cases, with properly distributed bracing members, a regularly shaped floor diaphragm as a whole may be considered as rigid in the horizontal plane. However, particular cases might require an analysis accounting for the in-plane deformability of the floor diaphragm.

The ties can be distributed along the edges of the precast floor elements or concentrated in the joints. Ordinary reinforcement or prestressing tendons in the precast floor units may be part of the tying system if properly connected across the joints by special tie arrangements. Special tying members can be incorporated in the floor system at intermediate joints or at the edges if anchored to the floor by proper tie arrangements.

Corners and voids of the diaphragm must be properly detailed so that the continuity of the tying system is assured.

When a cast in situ topping is provided, this should normally have a thickness on average not less than 40 mm, with minimum actual local thickness not less than 30 mm.

Vertical keys, suited for horizontal shear, may be of various shapes, dimensions and spacing; the depth is normally not less than 8 mm. Also proper artificial roughness of the joint interfaces can produce effects of indentations.

(e) Structural integrity

If the floor system is providing the integrity of the structure as a whole, precast floor elements should be directly or indirectly tied to the support at both ends.

The connections at the supports should be designed and detailed aiming at structural integrity and ductility in collapse situations. The connections should withstand large imposed deformations.

14.4.2. Specific design criteria

For specific types of precast floor units, deviations from the general code criteria can be permitted under certain conditions. Additional design criteria and minimum specifications on cross-section geometry, reinforcement, capacity and detailing of connections are given accordingly for each specific type of precast floor.

For the following types of precast floors, special rules for the design and detailing are presented in clauses 14.4.2.1 to 14.4.2.3

- floors with hollow core units
- composite floors with thin precast slab elements
- composite floors with precast rib and block systems.

14.4.2.1. Floors made of hollow-core units

(a) Support design

The bearing length should normally not be less than 55 mm. An even contact zone along the support should be provided.

The bearing length may be reduced at intermediate wall–floor connections if the design shear force is 50% or less of the shear capacity of the floor elements and the contact stress at the support is less than $0.25 f_{cd}$. However, the reduced value should not be less than 40 mm.

The design and detailing of the connections at the support should prevent a possible restraint initiating a shear failure near the support. Crack formation in the top layer should be prevented.

Shear design of units without shear reinforcement in the supporting zone should take into account the following modes of failure:

— shear compression failure;
— shear tension failure;
— anchorage failure of the main reinforcement.

Ties can be indirectly anchored to hollow core floor elements in concreted cores or in grouted joints, taking due account of reduced bond stresses that might arise from site practice and/or reduced cover in joints.

For anchorage in grouted joints, measures are required for the anchorage length of end anchors to make it possible to utilize the full strain capacity of the steel.

Axial tensile forces may occur due to restrained deformations at the support.

Models for verifying different failure modes are contained in FIP Recommendations for Hollow Core Slabs.

The tensile strength of concrete may be taken into account to determine the shear resistance on condition that

- sufficient load distribution capacity is available in the direction perpendicular to both the load and the span
- no significant axial tensile forces occur.

Any possible local damage has to be compensated by the redistribution capacity of the member itself or the structural system as a whole.

(b) Transverse load distribution

Transverse distribution is possible under the following conditions:

— longitudinal joints are able to transfer the vertical shear forces;
— a tying system prevents relative lateral displacement of adjacent precast units.

Under any rare combination of actions, the transverse flexural stresses occurring at the bottom of the precast elements should be limited to

$$\sigma_{ct,fl} < f_{ctk}/1.5$$

The transverse load distribution capacity can be calculated on the basis of slab theory assuming that the joints between slab units behave as linear hinges.

The reason for taking into account the axial tensile strength instead of the flexural tensile strength in the transverse direction is due to the hollow cores.

The ultimate load bearing capacity can be determined assuming that one longitudinal crack appears in the slab and cannot transmit any bending moment but only shear force, similar to the joints between the slabs.

The prefabricated plates are simply reinforced, or prestressed slabs. They are used as the shutter for the in situ concrete topping during construction.

Bonding reinforcement may be realized by ordinary stirrups, lattice trusses, ladders of reinforcement steel or adequate baskets.

Bonding reinforcement is necessary only in the areas where the limit shear stress can be exceeded. Thus, precast plates may have bonding reinforcement only near the bearings.

The transmission of shearing forces on a bearing may be realized by bars projecting from the edge of the plate, and bonded in in-situ concrete; if there are no projecting bars at the edge, this transmission may be achieved by stirrups close to the bearing zone.

A transverse connection across joints may be made with reinforcement bars placed within the cast in situ topping.

14.4.2.2. Composite floors with precast plates

For the design of composite floors, the completed floor can be considered monolithic, if the following conditions are fulfilled

- the method of construction of the precast plates ensures the roughness of the interface
- a bonding reinforcement is provided, with a correct anchorage in precast and cast in situ layers, if the design shear stress is higher than $0.3 f_{ctd}$ or if dynamic or impact loadings are to be considered
- a transverse connection is achieved at the joints between precast plates.

For floors with cores of non-structural materials, the following conditions should be met

- maximum spacing of ribs: 700 mm
- minimum thickness of precast plates and in situ concrete: 50 mm.

The bearing capacity and the deflections of the prefabricated plates during construction phases have to be verified by calculation or tests, taking into account the temporary supports.

Composite floors with precast ribs and blocks consist of partly prefabricated longitudinal ribs, intermediate prefabricated blocks or shutters and in situ concrete topping.

Part of the web and the topping are completed with in situ concrete. Infill blocks may function as simple shutter; in such a case, blocks typically have a modulus of elasticity

$$E_c < 8000\,\text{MPa}$$

A topping acts as a compression flange, if

$$h_0 \geqslant 40\,\text{mm and } i/12 \text{ for dwellings}$$
$$h_0 \geqslant 50\,\text{mm and } i/8 \text{ for other buildings,}$$

where h_0 is the thickness (i is the clear distance between ribs) and transverse reinforcement is provided consisting of at least $\phi 6$, $a \leqslant 330$ mm.

If infill blocks act as structural part, they typically have a modulus

$$8000 < E_c < 25000\,\text{MPa}$$

and shall provide all structural performance.

14.4.2.3. Composite floors with ribs and blocks

The transverse loadbearing capacity of this type of floor is based on shear transmission in the compression flange and on stiffening ribs in transverse direction. For spans exceeding 6 m, at least one transverse rib is required.

The width of ribs should be at least 50 mm.

If ribs are widened at the bottom to resist negative bending moments, the increase in the width is limited to 1 : 3 splay.

The spacing of ribs should not exceed 700 mm.

The bond capacity of the interface between the prefabricated part of the longitudinal rib and in situ concrete has to be verified in accordance with subsection 14.3.3.

Additional reinforcement in the anchorage zone of prestressing steel is generally required.

The ribs should be designed also for resisting loads applied before the hardening of in situ concrete. Otherwise they should be supported by intermediate props. Deflections at that stage should be checked.

Blocks have a minimum punching resistance of 1.5 kN with reference to a load applied on a 50 mm × 50 mm surface in the most unfavourable position.

14.5. WALL SYSTEMS

14.5.1. General

Wall systems consist of structural walls stiffened at each floor level by floors acting as horizontal diaphragms. The floors may be supported by the walls (load bearing wall system) or by beams and columns (dual system). In both cases the stability of the structures against horizontal actions is assured by the structural response of the wall system.

The structural requirements on overall integrity and robustness of the structure are to be met by tying the wall system together i.e. by providing the reinforcement at each floor level or similar arrangements, and by possible vertical ties.

When tensile stresses are expected in the wall, vertical tie reinforcement should be provided along the edges. This reinforcement crosses the horizontal joints and is an integral part of them.

Post-tensioning may be applied as well, to counteract the tensile stresses in the wall system.

When detailing the wall system, the difference in deformation and in displacement of the individual differently loaded wall portions (including deformations due to temperature and shrinkage) should be taken into account.

14.5.2. Structural analysis

Structural analysis of the wall system subjected to vertical and horizontal actions may be carried out separately for each action and the safety requirements may be checked for the sum of the resulting structural responses.

When the wall is subject to normal force combined with in-plane bending, the vertical strips as shown in Fig. 14.5.1 should be verified conventionally for compression, under the equivalent normal force

$$N_d = \sigma_d^* bt$$

where

t is the wall thickness

σ_d^* is the average design normal stress on the strip area tb.

Large deformations in the joints, exceeding those corresponding to the resistance of the joints (assumed for verifying the safety of the structures under normal actions) may be taken into account in the analysis of secondary stabilizing systems.

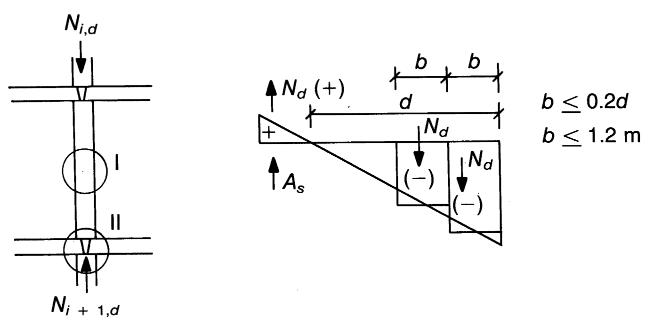


Fig. 14.5.1. Wall subject to normal forces and in-plane bending

The tensile part should be treated as a tensile member subjected to the action $N_d(+)$.

The above action effects are to be applied both to panels and joints.

The safety requirements regarding the normal force N_d acting in the wall are to be verified in two critical zones

zone I in the middle part of the wall, where the total eccentricity e_{tot} of the normal force N_d is affected by the second order eccentricity e_2 due to the slenderness of the wall,

zone II in the joint, where the wall resistance is reduced due to the disturbances of the forces.

Models for verifying the resistance of zone II are given in CEB Bulletin No. 169—'Draft Guide for the Design of Precast Walls Connections'.

The bearing capacity of the wall in zone I is to be verified according to the general rule for slender concrete members. In the verification (according to section 6.6) the additional eccentricity e_a covering execution inaccuracies should be taken into account.

The initial eccentricity e_0 of the normal force resulting from the vertical actions may be assumed *a priori* (hinged model of the wall systems) or calculated making use of a continuous frame model. The restraint to bending of floor–wall joints is to be taken into account in the second case.

14.6. BEAM AND COLUMN SYSTEMS

14.6.1. General

Building structures made of linear precast elements (beams and columns) may be designed in order to meet one of three basic structural schemes

(*a*) continuous framework
(*b*) cantilevered continuous columns
(*c*) 'hinged joints' plus bracing walls.

Bracing walls may also be used combined with schemes (*a*) and (*b*) (dual schemes).

Normally, all schemes are associated with horizontal floor decks acting as diaphragms.

The behaviour of the three schemes differs essentially in the distribution of horizontal actions.

Scheme (b) is normally associated with low-rise buildings such as industrial halls. Often floor connections in the roof provide only axial (no in-plane shear) action.

The conventional lateral force as overall means for verifying stability is assumed to be 1% of vertical loads acting at each floor.

Regularity of layout and dispersed bracing is recommended.

Structural integrity and limitation of damage propagation after accidental events should be provided depending on the particular layout (see sub-section 14.1.2).

14.6.2. Structural analysis

Continuity may be assumed for connections concreted in situ and fully respecting the detailing of monolithic frame joints or for connections that have been proved continuous by means of accurate testing of resistance and stiffness under unfavourable conditions.

Testing of connections for rigid beam-column joints should consider effects of repeated and alternate loadings, of internal actions (shrinkage and temperature constraints), and possible degradations. Seismic requirements are not included in this code.

Connections should otherwise be considered as hinged in the analysis related to lateral actions, and for the assessment of a realistic stiffness for other actions.

Particular attention should be paid in diaphragm detailing of structures pertaining to scheme (*c*).

In case of absence or discontinuity of diaphragm (e.g. hall roofs with simply supported, non-mutually connected elements), effects of displacement on stability members and compatibility of bearings should be verified.

14.7. SEGMENTAL CONSTRUCTION

The deformation line of the assembled structure is to be computed and checked at every construction stage.

This is a method of assembling precast segments by means of post-tensioning.

14.7.1. Joints

14.7.1.1. Orientation

In principle the middle surface of a joint should be perpendicular to the post-tensioning force.

An inclination of less than 20° is acceptable if the shape of the surface is such that the existence of sufficiently high friction forces can be counted on.

14.7.1.2. Types of joints

Two types of joints are commonly used

- wide joints (with cast in situ concrete, dry-pack mortar, or grout joint filler)
- match cast joints (with epoxy bonding agent or dry joints), including wide joints made to match at a later stage.

14.7.1.3. Wide joints

The width of cast in situ joints should permit effective vibration of the concrete and allow for any necessary welding of reinforcement or of metal inserts.

The strength of the joint material should be not less than 25 MPa or that of the joined segments.

The recommended approximate widths are

$\geqslant 80$ mm for cast in situ joints
$\geqslant 25$ mm for tamped joints
$\leqslant 15$ mm for grouted joints

The obtained accuracy of structure depends mainly on the preparation and casting of the joints during erection rather than on the precision of the segments themselves.

14.7.1.4. Match cast joints

The compressive strength of the material of the joint should be at least equal to that of the concrete of the joined segments.

If necessary, consideration should be given to the creep of this material.

The joint surfaces fit each other perfectly because each element is cast against those with which it will be in contact in the structure. The surface can comprise indentations, which ensure correct positioning when they are erected and adequate resistance to load effects.

The recommended width of the epoxy resin layer is 1–3 mm.

14.7.2. Structural analysis

When calculating the load effects and strength and drawing up the detailed design, the structural components obtained by these methods may be considered as monolithic for serviceability limit states.

No tension is acceptable in the joints under rare combinations of actions.

The shear in joints without shear keys (wide joints only) should satisfy $V_{Sd} \leqslant V_{Rd}$.

For verifying limit states of decompression, an appropriate temperature gradient between opposite layers of the assembled structure should be accounted for.

The characteristic values of μ_k can be found by tests.

V_{Rd} may be calculated according to clause 14.3.3.1.

The shape of the keys may be chosen to suit the particular application; they can be broadly distinguished into two categories

- single keys, generally large and localized
- multiple keys, generally covering as much of the joint surface as practical.

Shear keys located in the compressive zone under bending (ULS) are of major importance.

Appendices

APPENDIX A

NOTATION

The aim of this appendix is merely to describe the symbols to be used without prejudging the exact definition of each term.

Notation should be in accordance with ISO 3898.

a.1. Construction of symbols

A symbol to represent a given quality or term is constructed as follows.

(1) The main letter of the symbol is chosen from a.2, a.3, a.4, or a.5 on the basis of its dimensions and its use, as given in Table a.1.
(2) Descriptive subscripts may be chosen at will. When subscripts other than those appearing in a.6, a.7 and a.8 are used, a clear definition of their meaning shall be given.
(3) In constructing symbols, the first subscripts shall indicate the location and the following ones the cause (nature, location, etc.). When necessary to avoid confusion, the use of a comma between the two categories of subscript is recommended.
(4) When there is no risk of confusion, all or some of the descriptive subscripts, may be omitted.
(5) Numbers may be used as subscripts if necessary.
(6) An apostrophe (') representing compression is to be added to symbols representing geometrical quantities, if necessary.

Table a.1. Guide for the construction of symbols

Type of letter	Dimensions	Use
Roman capital	Force; force times length; length to a power other than 1; temperature	1. Actions and action effects; work; energy 2. Area; volume; first and second moments of area 3. Temperature 4. Moduli of deformation (exception to the general rule)
Roman lower case	Length; quotient of length and time to a power; force per unit length or area; except when used as a subscript	1. Linear dimensions (length, width, thickness, etc.) 2. Velocity; acceleration; frequency 3. Actions and action effects per unit length or area 4. Strengths 5. Descriptive letters (subscript) 6. Mass 7. Time
Greek capital	—	Reserved for mathematical symbols
Greek lower case	Dimensionless	1. Coefficients and dimensionless ratios 2. Strains 3. Angles 4. Densities (related to mass or weight) (exception to the general rule) 5. Stresses

Note: concepts not included in Table a.1 should be classified in the nearest category.

To avoid confusion, the following precautions should be taken.

(*a*) The possibility of confusing 1 (numerical) with l (letter) in some typed documents has been recognized. L will therefore be used in place of l (letter) when there would be a risk of ambiguity in typed documents.

(*b*) Roman upper and lower case letter O shall not be used as a main letter owing to the possibility of confusion with other symbols. For the same reason, it is recommended that kappa (κ) and chi (χ) should be avoided as far as possible. Lastly, if the lower case Greek letters eta (η) and omega (ω) are used, care must be taken in writing them to avoid confusion with the lower case Roman letters n and w.

a.2. Meaning of Roman capital letters*

A area
B
C torsional moment of inertia
D fatigue damage factor; diffusion coefficient
E modulus of elasticity; earthquake action
F action in general; local loading
G permanent action; shear modulus
H horizontal component of a force
I second moment of a plane area
J creep function
K (permeability) coefficient
L can be used for 'span; length of an element' in place of l
M bending moment; coefficient of water absorption
N axial force
O (void)
P prestressing force
Q variable action
R strength (resisting load effect); reaction at a support; resultant
S load effect (M, N, V, T); static moment of a plane area
T torsional moment; temperature
U
V shear force, volume
W modulus of inertia
X reaction or force in general, parallel to x-axis
Y reaction or force in general, parallel to y-axis
Z reaction or force in general, parallel to z-axis

* Roman capital letters can be used to denote types of material, e.g. C for concrete, LC for lightweight concrete, S for steel, Z for cement.

a.3. Meaning of Roman lower case letters

a deflection; distance; acceleration
b width
c concrete cover
d effective height; diameter (see also h and a.5)
e eccentricity (see also a.5)
f strength of a material
g distributed permanent load; acceleration due to gravity
h total height or diameter of a section; thickness
i radius of gyration
j number of days
k all coefficients with dimension
l span; length of an element

m	bending moment per unit length or width; mass; average value of a sample
n	normal (longitudinal, axial) force per unit length or width
o	(void)
p	(void)
q	distributed variable load
r	radius
s	spacing; standard deviation of a sample
t	time; torsional moment per unit length or width; thickness of thin elements
u	perimeter
v	velocity; shear force per unit length or width
w	width of a crack
x	co-ordinate; height of compression zone
y	co-ordinate; height of rectangular diagram
z	co-ordinate; lever arm

a.4. Use of Greek lower case letters

alpha	α	angle; ratio; coefficient
beta	β	angle; ratio; coefficient
gamma	γ	safety factor; density; shear strain (angular strain)
delta	δ	coefficient of variation; coefficient
epsilon	ε	strain
zeta	ζ	coefficient
eta	η	coefficient
theta	θ	rotation
iota	ι	(void)
kappa	κ	(to be avoided as far as possible)
lambda	λ	slenderness ratio; coefficient
mu	μ	relative bending moment; coefficient of friction; mean value of a whole population
nu	ν	relative axial force; Poisson's ratio
xi	ξ	coefficient; ratio
omicron	o	(void)
pi	π	(mathematical use only)
rho	ρ	geometrical percentage of reinforcement; bulk density
sigma	σ	axial stress; standard deviation of a whole population
tau	τ	shear stress
upsilon	υ	(void)
phi	ϕ	creep coefficient
chi	χ	(to be avoided as far as possible)
psi	ψ	coefficient; ratio
omega	ω	mechanical percentage of reinforcement

a.5. Mathematical symbols and special symbols

Σ	sum
Δ	difference; increment (enlargement)
$\varnothing$	diameter of a reinforcing bar or of a cable
'	(apostrophe) compression (only in a geometrical or locational sense)
e	base of Naperian logarithms
exp	power of the number e
π	ratio of the circumference of a circle to its diameter

n	number of . . .
w/c	water/cement ratio
$\not>$	not greater than: * indicates the upper bound in a formula
$\not<$	not smaller than: * indicates the lower bound in a formula
$<$	smaller than
$>$	greater than

* These symbols placed at the end of an expression indicate that where the result to which it leads is higher (or lower) than the limit given, then the values given should be taken into account and not the result obtained from the formula.

a.6. General subscripts

a	support settlement; additional; accidental load
b	bond; bar; beam
c	concrete; compression; column
d	design value
e	elastic limit of a material
f	forces and other actions; beam flange; bending; friction
g	permanent load
h	horizontal; hook
i	initial
j	number of days
k	characteristic value
l	longitudinal
m	mean value; material; bending moment
n	axial force
o	zero
p	prestressing steel
q	variable load
r	cracking
s	ordinary steel; snow; slab
t	tension;* torsion;* transverse
u	ultimate (limit state)
v	shear; vertical
w	wind; web; wire; wall
x	linear co-ordinate
y	linear co-ordinate
z	linear co-ordinate
1, 2, 3 . . .	particular values of quantities
∞	conventional asymptotic value

* When confusion is possible between tension and torsion, the subscripts *tn* (tension) and *tr* (torsion) should be used.

a.7. Subscripts for actions and action effects

$a(A)$	support settlement; accidental action
cc	creep of concrete
cd	delayed elasticity of concrete
cf	delayed plasticity of concrete
cs	shrinkage of concrete
ep	earth pressure
$eq(E)$	earthquake; seismic
ex	explosion; blast
$f(F)$	forces and other actions
$g(G)$	permanent load
im	impact
lp	liquid pressure

$m(M)$	bending moment
$n(N)$	axial force
$p(P)$	prestress
$q(Q)$	variable load
$s(S)$	snow load
$t(T)$	torsion; temperature
$v(V)$	shear
$w(W)$	wind load

a.8. Subscripts obtained by abbreviation

abs	absolute
act	acting
adm	admissible, permissible
cal	calculated, design
crit (or *cr*)	critical
ef	effective
el (or *e*)	elastic
est	estimated
exc	exceptional
ext	external
fat	fatigue
inf	inferior
int	internal
lat	lateral
lim	limit
max	maximum
min	minimum
nec	necessary
net	net
nom	nominal
obs	observed
pl	plastic
prov (or *pr*)	provisional (stage of construction), provided
red	reduced
rel	relative, relaxation
rep	representative
req	required
res	resisting, resistant
ser	serviceability, service
sup	superior
tot	total
var	variable

APPENDIX B
TERMINOLOGY ON CONSTRUCTION WORKS

In order to avoid some ambiguities which are common in the usual language, it is recommended to use the following terms in accordance with the definition given below.

- *Construction works*: Everything that is constructed or results from construction operations. In accordance with ISO 6707 Part 1, this term covers both building and civil engineering works. It refers to the complete construction comprising both structural and non-structural elements.
- *Execution*: The activity of creating a building or civil engineering works. The term covers work on site; it may also signify the fabrication of components off site and their subsequent erection on site.
- *Structure*: Organized combination of connected parts designed to provide some measures of rigidity. This term refers only to load carrying parts.
- *Type of building or civil engineering works*: Type of 'construction works' designating its intended purpose, e.g. dwelling house, industrial building, road bridge.
- *Form of structure*: Structural type designating the arrangement of structural elements, e.g. frame, beam, triangulated structure, arch, suspension bridge.
- *Construction material*: A material used in construction work, e.g. concrete, steel, timber, masonry.
- *Type of construction*: Indication of principal structural material, e.g. reinforced concrete construction, steel construction, timber construction, masonry construction, composite construction.
- *Method of construction*: Manner in which the construction will be carried out, e.g. cast in place, prefabricated, cantilevered.
- *Structural system*: The load-bearing elements of a building or civil engineering works and the way in which these elements are assumed to function, for the purpose of modelling.

APPENDIX C
DESIGN BY TESTING

This appendix has a provisional character. It may be amended when the relevant basic documents of the JCSS become available.

c.1. Scope

This appendix contains guidance for the experimental assessment of the response of structural members not included in the Model Code's range of application. In very particular cases only (see section c.3) the content of this appendix may be used in order to modify Code provisions.

In this context 'response' may mean deformation or strength characteristics.

Examples of such structural members may be

- assemblages of reinforced concrete prefabricated elements
- units with special shape (polyhedric panels, bunkers).

Examples of special design problems studied experimentally may be

- prestress losses
- special production processes (accelerated curing etc.).

However, this appendix covers only those cases in which a statistical interpretation of results is possible.

Examples of tests which generally cannot be statistically interpreted

- tests on frictional losses of prestress made by measurements of forces transmitted by tendons on the structure during construction
- tests on uncertainties related to static equilibrium made by measurements on the smaller reaction of supports.

This appendix does not contain information about experimental investigation used for structural analysis.

The development of computer calculations has considerably reduced the interest of tests to this purpose.

Tests to be made for Agrément would need further developments and are not fully covered by this appendix.

c.2. Definition

Design by testing (DbT) is considered here as a procedure where loading tests on limited series of representative specimens are used for the determination of the response of structural members.

In a wide sense design by testing may cover all kinds of load tests on either scaled-down models, on special specimens or on the entire structure itself; these tests are expected to give additional information to the designer.

In this respect, two particular cases may be distinguished in practice

(*a*) *Small structural elements repeatedly produced under factory conditions.* An improvement of the existing design model is sought; a considerable number of full scale specimens are tested, taking into account all actions and influences during the lifetime of the structure where these elements will be incorporated.

(*b*) *Larger structural elements to be constructed under site conditions.* An analytical model may not be initially available or considerable modification of existing models is sought. A comparatively lower number of tests are carried out simulating the real life conditions. This case is meant to be the main case addressed by this appendix.

However, the following cases are not covered by this appendix:

- non-destructive test-loadings of single finished structures
- wind tunnel or earthquake simulator tests
- routine control tests on reinforced concrete industrial products
- accelerated durability tests.

c.3. Aims of design by testing

The aim of the design by testing is to obtain design values for the response of structural members under specified load conditions with respect to a certain limit state.

Design by testing may be pursued instead of design by calculation when

(*a*) the calculation models are insufficient or out of the Code's range of application

Such cases may be deviations from the provisions of the present Code with respect to, for example

- geometric shapes
- combination of new materials
- detailing and/or connections
- severe actions.

(*b*) design by testing may also be used in some particular cases and with special care, either when the calculation models given in the present Code are thought to lead to uneconomic results, or where greater precision is desirable.

Such cases may be considered where, for example

- the economic importance of a project or of an element to be mass produced justifies the effort of design by testing
- the codified model, in some particular conditions is judged to be clearly too rough.

c.4. Requirements

The design-by-test procedure should ensure that the design leads to the same level of reliability as the Code.

To this end all possible conditions, actions and possible influences expected during the lifetime of the structure shall be appropriately reproduced in laboratory.

The whole procedure shall be developed by the designer and be approved by the relevant counterpart.

The experimental assessment shall be performed by qualified institutions with staff experienced in planning, executing and evaluating tests.

Special care is necessary for several reasons.

- Computational design versus ultimate limit states often covers indirectly serviceability as well and intermediate limit states (which for simplicity are not codified): a modification of the codified provisions for ULS might result in deficiencies in this respect.
- Design by testing is not able to account for some uncertainties (e.g. on structural analysis or on parasitic phenomena due to the environment) which are covered by code provisions, nor to justify a reliability format more precise than the codified one.

This is particularly important when design by testing is to be permitted for financial reasons (see section c.3).

In this respect, see *inter alia*

(*a*) the in-time variation of the basic variables (subsection c.5.2)
(*b*) the condition affecting the conversion factor η (clause c.9.1.1).

If some of these variations and conditions are expected to have a systematic effect on the structural response under investigation, their complete reproduction in the laboratory shall be secured.

c.5. Planning

c.5.1. Calculation model—limit states

A plan should be drafted by the designer, which shall contain the objective of testing and all indications necessary for the sampling or manufacturing of the specimens, the execution of the tests and their evaluation.

The experimental procedure may cover either an ultimate limit state or a serviceability limit state. In either case, a minimum knowledge of the relevant response mechanism and parameters is sought. It is desirable that on the basis of this knowledge, an empirically or physically based calculation model is available prior to testing, in order to evaluate the response. This model is referred to as 'prior calculation model' g_R

If such a model is not available prior to testing, trial preliminary tests should be carried out in order to facilitate parameter identification.

$$R_t = g_R(\boldsymbol{X}, \boldsymbol{W}, \boldsymbol{D}) \qquad (c\text{–}1)$$

where

The response depends in general on a set of measurable quantities. The quantities which are random with respect to an elementary population are referred to as basic variables $\boldsymbol{X}$. The quantities which are considered as deterministic, with respect to an elementary population, are referred to as nominal variables $\boldsymbol{W}$. Nominal variables may be constant within a population or vary in a predetermined manner.

The terminology used in this clause and the following ones, with regard to the variables, is not exactly the same as in Bulletin 191 and in chapter 1 (section 1.3 and subsequent) of this Model Code because the character of some variables (fundamental or not) is not yet established at this stage. Some variables indeed, although random, may have very little influence on the response.

In case of unidentified significant parameters, a larger scattering of results is expected.

Special care is necessary because statistical parameters of basic variables have not an intrinsic character. They generally depend on the considered statistical population, which may be very different at the level of the Code, at the level of the specimens and at the level of the expected application of the test result.

Special mention should be made of the quasi-deterministic concrete strength of the specimens which exhibit a very low variability of concrete strength. Thus, in the relevant model, the characteristic strength of in-structure concrete should be made equal to the characteristic strength of the specimen concrete.

For example

- in-time development of the concrete strength positive or negative because of some additives
- possible decrease of the tensile strength of concrete due to hydro-thermal cyclic conditions
- possible decrease of steel ductility or fatigue performance due to minor corrosion conditions.

The necessary number of tests is higher where not only mean values but also standard deviations are to be statistically assessed.

R_t is the response of the available analytical model
$\boldsymbol{X}$ is the vector of basic variables
$\boldsymbol{W}$ is the vector of nominal variables
$\boldsymbol{D}$ is the vector of unknown coefficients to be determined by the testing.

All quantities affecting the response must be present in the model either as basic variables or as nominal variables.

c.5.2. Information on basic variables

For the basic variables included in the calculation model, statistical parameters have to be known. If these values are not known, it is recommended to estimate them by preliminary tests. Where information is available only from a limited population, variances need to be increased accordingly.

The in-time variation of the basic variables should be taken into account in the model.

Gross-errors cannot be covered by this procedure.

c.5.3. Number of specimens

The number of tests carried out should be sufficiently large in order to lead to results with satisfactory small confidence interval (see section 8.3).

c.5.4. Scale effects

The specimens should preferably be dimensioned as close to full scale as possible, so that the scale effects do not increase the model uncertainties. Otherwise, similitude laws and fracture mechanics shall be appropriately taken into account.

The dimensions of the specimens should, if possible, cover the entire range of the probable variation of the dimensions of the structural element.

When scaled-down specimens are used, special attention should be paid to the influence of aggregate size, re-bars size, concrete workability etc. on the response.

c.5.5. Actions

The actions may be direct forces (loads in general), or imposed deformations or other influences varying in space and/or in time. The scheduled actions process must be fully determined by a set of load (or occasionally of other influence) parameters. The actions process shall be selected so that they are representative for the anticipated scope of application of the structural member.

Each specimen may be subjected to the same or different action processes.

Other influences may be temperature, humidity etc., conditioning the behaviour of the specimens and their response.

The actions process ('load path') should also include data regarding rate effects (e.g. load application velocity, the order of application of each influence).

c.5.6. Origin of specimens

The specimens should be specifically manufactured for the testing.

Specimens taken from production may be used either for preliminary tests or in the case (a) defined in section c.2.

c.6. Testing conditions and measurements

Special care should be taken in order to check that all assumptions made in the planning (see section c.5) are satisfied.

Quantitative and qualitative observations are highly recommended in order to check these assumptions, made implicitly or explicitly, or to allow alternative evaluations afterwards.

If, during testing, a given condition is found to influence considerably the results, this condition shall be more systematically studied as a new basic variable of the model g_R.

c.6.1. Basic and nominal variables

The actual values of all basic and nominal variables included in the calculation model should be, as far as practicable, determined by direct measurements for each specimen and each experiment.

When a direct measurement is not possible, an indirect measurement is allowed. Then the respective conversions should be used, by introducing the necessary new variables of the conversion factors or by increasing the variance of the relevant variable.

In the case of indirect measurement of a variable, the difference of the statistical properties between the populations in the laboratory and on site shall be taken into account.

c.6.2. Actions

The actual values of the imposed actions have to be recorded during the test and especially at the 'critical point' of the limit state considered.

Since the response of the structural member is in general identified in terms of load intensity at which the limit state is reached, the actual limit value of the limit load should be measured with additional care.

c.6.3. Deformation—structural behaviour

During the tests, systematic measurements shall be carried out concerning the deformations (elongation, deflection, rotation).

It is recommended that measurements are redundant in order to make a mutual checking of the results possible.

c.7. Laboratory report

The laboratory report should contain at least

- the name of the part asking for the design by testing
- the names of the laboratory staff involved in the DbT
- the scope of the DbT
- technical description of the specimens (dimensions, materials, fabrication technique, number of specimens, etc.)
- testing procedure
- all measurements of basic and nominal variables
- the actions process
- the deformation of specimens
- crack pattern
- the failure mode and the critical material
- photos and/or video recordings
- the prior calculation model used
- laboratory comments on the obtained results.

Further useful guidance on the matter may be found in the Recommendation of RILEM TC 125.

c.8. Statistical analysis of test results

c.8.1. Estimation of the coefficients *D*

The estimation of the coefficients D (eq. (c–1)) may be based on

- least square methods
- maximum likelihood methods.

The coefficients D may also be considered as random variables, but in order to check the validity of the equation (c–1), in a first step, the coefficient D should be given the best value D_m.

For linear models or models which can be transformed to linear, coefficients generally should be assumed as normally distributed.

Where no linear models are obtained, a Taylor expansion in the vicinity of the expected design point may be used to avoid non-linear regression analysis.

For linear models of the type

$$Y = d_1 x_1 + d_2 x_2 + \ldots + d_n x_n = \boldsymbol{DX'}$$

the joint distribution of D is a multivariate central t-distribution. For details, reference is made to the relevant literature.

When coefficients of mechanical models are to be estimated, prior information on these coefficients should be derived from physical considerations.

Often it is known that coefficients can only take values larger or smaller than unity.

c.8.2. Correlation between experimental and theoretical values

In general, the observed/experimental values R_e of the response will be different from the corresponding theoretical values $R_t = g_R(\boldsymbol{X}_m, \boldsymbol{W}, \boldsymbol{D}_m)$ (predicted by the model). This scatter is usually measured by the correlation coefficient ρ $(0 \leqslant \rho \leqslant 1)$.

When the correlation coefficient is high the correlation is considered to be sufficient.

$\rho = 0.85$ to 0.90 is considered to be a sufficient value.

High correlation does not necessarily imply a good model. In fact, if the number of unknown parameters tends to the number of observations, then ρ tends to unity. This is for example the case when for one basic variable, a polynomial of $(n - 1)$ degree has been considered as prior calculation model

$$Y = d_0 + d_1 x + d_2 x^2 + \ldots + d_{n-1} x^{n-1}$$

where n is the number of test results.

In such a case $\rho = 1$.

A graphic representation of the differences $(R_e - R_t)$ as a function of R_t is necessary to verify the validity of the prior calculation model.

c.8.3. Characteristic value

The characteristic value $R_k = g_R(\boldsymbol{X}_k, \boldsymbol{W}, \boldsymbol{D}_k)$ (5% fractile or other) can be calculated from test results by means of appropriate statistical analysis.

The characteristic value R_k generally is defined as the value for which

$$P(R_t < R_k) = 0.05$$

with a level of confidence that normally is taken about 0.75 (unilateral limit).

c.9. Design procedure

c.9.1. Design values

c.9.1.1. Design values for ULS

The design value of the response is given by the expression

$$R_d = \eta R_k / \gamma_{Rd} \gamma_m$$

where

R_k is the characteristic response defined statistically on the basis of the test results

γ_m is the material partial safety factor adopted according to the failure mode of the material decisive for the bearing capacity

If relevant, the characteristic values X_k are substituted in R_k by other representative values depending on the combination and/or the limit state.

The material safety factor γ_m is to be adopted in accordance with this Code as

$$\gamma_m = \gamma_s \quad \text{or} \quad \gamma_m = \gamma_c$$

see section 1.6 for numerical values of the γ-factors according to the failure mode and to the material decisive for the failure.

η is a conversion factor, taking into account the differences between testing conditions and conditions in the actual structure.
If a more detailed model is known, accounting separately for the role of both materials, separate conversion and safety factors may be used accordingly.

γ_{Rd} is a complementary model-uncertainty-factor intended to cover differences between the testing conditions and the actual ones in the structure, which cannot be accounted by D_k, by γ_m or by the conversion factor η.

The differences covered by the factor η may, for example, be

- loading time (when concrete strength is critical, a factor equal to 0.85 should be applied at least to account for sustained loads)
- support conditions
- humidity conditions and their alternations
- differences in geometry
- differences in workmanship and/or curing conditions.

Influences of systematic character (e.g. a brittleness factor such as $1 - f_{ck}/250$) should be covered by the model and not by η.

Those conversion conditions included in γ_m should not be duplicated when assessing η.

Only influences of secondary importance may be accounted for by means of conversion factors estimated in a more or less empirical way.

The basic value of γ_{Rd} results from a comparison to be made between $D_k\eta/D_m\gamma_m$ adopted above with the η/γ_m given in the Code in a similar case of design by calculation. γ_{Rd} should be increased according to experience in cases where the scatter of D is high (e.g. its coefficient of variation is greater than 0.15) or the expected variability of the basic variables is significantly higher on site than in specimens (see subsection c.5.2), or the value of the conversion factor on site conditions seems to be less than the already adopted η value.

Experts' opinions may be used in evaluating η and γ_{Rd}, provided that all systematic differences are taken into account in the tests.

γ_{Rd}-values for SLS are generally different than for ULS; some minor aspects of a conversion factor may as well be accounted for by this γ_{Rd} factor.

c.9.1.2. Design values for SLS

The design values for the serviceability limit states are given by the expression

$$R_d = R_k/\gamma_{Rd}$$

with R_k and γ_{Rd} as defined in clause c.9.1.1.

c.9.2. Verification

The most frequent verification inequality is

$$S_d \leqslant R_d$$

where

S_d denotes the design value of the load effects
R_d denotes the design value of the the response.

Basic documents

MENEGOTTO, M. Justification of structural components and assemblages by testing—Relationship with analytical code design. FIP Commission on Prefabrication. April 1983.

THORENFELDT, E. Design by testing of concrete structures; CEB Commission I.

TASSIOS, T.P. Design based on testing; Scientific Papers, Faculty of Civil Engineering, N.T.U.A., Vol. 2, April–June 1983.

CEB/ECCS. Note No. 10 'General rules for design by testing', July 1985.

CEB/ECCS. Note No. 11 'Evaluation of test results', July 1985.

LEWICKI, B. 'Some remarks in connection with the anticipated reviewing of the Model Code 1978', CEB Commission I, April 1985.

LEWICKI, B. 'Example to Note No. 11', 'Evaluation of test results', August 1986.

MAIER, W. 'Design by Testing', April 1986.

EC3. 'Background document for Chapter 9 of Eurocode 3', July 1988.

JCSS. 'Estimation of structural properties by testing for use in limit state design', August 1989.

LEWICKI, B. 'Design by testing', ISO, TC98, SC2, November 1989.

APPENDIX D
CONCRETE TECHNOLOGY

d.1. Scope

This appendix gives information on the technical requirements to be satisfied by the constituent materials of concrete, concrete composition, the properties of fresh and of hardened concrete, the production, placing and curing of concrete and the verification of the required properties. Concrete which satisfies these requirements will have mechanical and durability properties generally in accordance with the properties on which the design rules of this Model Code are based. This appendix does not deal with special concretes such as fibre concrete, heavyweight concrete, polymer concrete, repair mortars and organic compounds other than admixtures. However, it includes information on lightweight aggregate concrete and on concrete with a compressive strength in excess of 50 MPa, often referred to as high strength concrete.

d.2. Reference documents

This appendix should be interpreted in conjunction with other international documents such as the standards and recommendations prepared by

CEN:	European Committee for Standardization
ISO:	International Organization for Standardization
RILEM:	International Union of Testing and Research Laboratories for Materials and Structures

Particular attention is drawn to the following documents though other national or international standards may be of equal relevance.

prENV 197	Cement; composition, specifications and conformity criteria, June 1989.
ENV 206	Concrete—Performance, production, placing and compliance criteria.
ISO 1920	Concrete tests—Dimensions, tolerances and applicability of test specimens.
ISO 2736/1	Concrete tests—Making of test specimens—Part 1: Sampling of fresh concrete.
ISO 2736/2	Concrete tests—Making of test specimens—Part 2: Making and curing of test specimens for strength tests.
ISO 4012	Concrete—Determination of compressive strength of test specimens.
ISO 4013	Concrete—Determination of flexural strength of test specimens.
ISO 4108	Concrete—Determination of tensile splitting strength of test specimens.
ISO 4109	Fresh concrete—Determination of the consistency—Slump test.
ISO 4110	Fresh concrete—Determination of the consistency—Vebe test.
ISO 4111	Fresh concrete—Determination—Degree of compactibility (Compaction index).
ISO 4848	Concrete—Determination of air content of freshly mixed concrete—Pressure method.
ISO 6275	Concrete, hardened—Determination of density.
ISO 6276	Concrete, compacted fresh—Determination of density.
ISO 6782	Aggregates for concrete—Determination of bulk density.

ISO 6783	Coarse aggregates for concrete—Determination of particle density and water absorption—Hydrostatic balance method.
ISO 7031	Concrete, hardened—Determination of the depth of penetration of water under pressure.
ISO 7033	Particle density and water absorption of fine and coarse aggregates for concrete (pycnometer method) (at present at the stage of draft standard).
ISO 7034	Cores of hardened concrete—Taking, examination and testing in compression (at present at the stage of draft standard).
ISO 8045	Concrete, hardened—Determination of rebound number using the rebound hammer (at present at the stage of draft standard).
ISO 8047	Concrete, hardened—Determination of ultrasonic pulse velocity (at present at the stage of draft standard).
ISO 9812	Fresh concrete—Determination of consistency—Flow test (at present at the stage of draft standard).
RILEM CPC7	Direct tension (final recommendation, 1975).

In this document frequently reference is made to 'Durable Concrete Structures—CEB Design Guide' CEB Bulletin d'Information No. 182, 1989.

d.3. Definitions

Some of the technical terms relating to concrete technology and used in this appendix are defined as follows.

(1) *Concrete*: Material formed by mixing cement, coarse and fine aggregate and water and produced by the hydration of the cement paste (cement and water); in addition to these basic components, it may also contain admixtures and/or additions. If the maximum particle size of the aggregate is 4 mm or less, the resulting material is generally termed mortar, not concrete.

(2) *Fresh concrete*: Concrete still in the plastic state and capable of being compacted by normal methods.

(3) *Hardened concrete*: Concrete which has hardened and developed strength.

(4) *Site mixed concrete*: Concrete batched and mixed on or near the construction site by the user.

(5) *Ready-mixed concrete*: Concrete batched in a plant outside or on the construction site, mixed in a stationary mixer or a truck mixer and delivered by the producer to the user as fresh concrete ready for use either on the construction site or into a vehicle of the user.

(6) *Batch*: The quantity of concrete mixed in one cycle of operations of a batch mixer, or the quantity of concrete conveyed as ready-mixed concrete in a vehicle, or the quantity discharged during approx. 1 min from a continuous mixer.

(7) *Normal weight concrete*: Concrete having an oven-dry (105°C) density greater than 2000 kg/m^3 but not exceeding 2800 kg/m^3.

(8) *Lightweight aggregate concrete*: Concrete having an oven-dry density of not more than 2000 kg/m^3. It is entirely or partly produced by the use of aggregate that has a porous structure (lightweight aggregate).

(9) *Heavyweight concrete*: Concrete having an oven-dry density greater than 2800 kg/m^3.

(10) *Admixture*: Product which is added in quantities generally less than or equal to 5% by mass of the cement before or during mixing or during an additional mixing operation, causing the required modifications to the normal properties. In special cases amounts > 5% may be added.

(11) *Addition*: Finely divided inorganic material that may be added to the concrete in order to improve certain properties or to achieve special properties. There are two types of inorganic additions: nearly inert additions (Type I) and pozzolana or latent hydraulic additions (Type II).

(12) *Aggregate*: Concrete component consisting of uncrushed and/or crushed natural and/or artificial mineral substances with particle sizes and shapes suitable for the production of concrete.

(13) *Normal weight aggregate*: Aggregate with a particle density between 2000 and 3000 kg/m^3, the particle density being determined, for example, according to ISO 6783 or ISO 7033.

(14) *Lightweight aggregate*: Natural or artificial aggregate consisting of particles with a porous structure and with a particle density less than 2000 kg/m^3, the particle density being determined, for example, according to ISO 6783 or ISO 7033.

(15) *Heavyweight aggregate*: Aggregate having a particle density larger than 3000 kg/m^3, the particle density being determined, for example, according to ISO 6783 or ISO 7033.

(16) *Cement*: A hydraulic binder, i.e. a finely ground inorganic material which when mixed with water, forms a paste which sets and hardens by hydration reactions and processes and which, after hardening, retains its strength and stability even under water.

(17) *Effective water content*: Mixing water plus water already present on the surface of the aggregates or in the admixtures and additions.

(18) *Water/cement ratio*: Ratio of effective water content to cement content of the concrete (refer also to clause d.6.3.3.2).

(19) *Entrained air*: Microscopic air bubbles intentionally incorporated in concrete during mixing, usually by use of surface active agents; typically between 10 and 1000 μm in diameter and spherical or nearly so.

(20) *Entrapped air*: Air voids in concrete which are not purposely entrained and which are significantly larger and less useful than those of entrained air, 1 mm or larger in size.

(21) *Designed mix*: A mix for which the user specifies the required performance of the concrete and any additional characteristics, and the producer is responsible for providing a mix which complies with the required performance and additional characteristics.

(22) *Prescribed mix*: A mix for which the user specifies the composition of the mix and materials to be used. The producer is responsible for providing the specified mix but is not responsible for the performance of the concrete.

(23) *High strength concrete*: Concrete with a characteristic strength higher than 50 MPa.

d.4. Constituent materials

d.4.1. General requirements

The basic materials (i.e. cement, aggregates, water, admixtures and additions) shall be suitable for making concrete which will attain and retain the required properties. Therefore, the materials should meet certain requirements as to composition and physical properties. Moreover, their quality should be consistent, and no impurities shall be introduced during transport and storage.

Even small traces of foreign materials such as gypsum, lime, zinc or sugar can have an adverse effect on the essential properties of concrete, such as the setting, hardening or dimensional stability. Constituents shall not contain ingredients which may cause corrosion of embedded reinforcement. Only such basic materials should be used whose properties are verified by an appropriate certification body and which are subject to continuous quality control with the exception of water. General instructions for the choice and evaluation of the constituent materials are given in the following. Further information may be sought in international and national standards or regulations.

d.4.2. Cement*

d.4.2.1. Types and requirements

The cements shall satisfy the requirements of national or international standards. The most important standards for all applications are those related to setting behaviour, strength development, dimensional stability and, for some special applications, heat of hydration and resistance to internal and external chemical attack.

The following classification is widely accepted and proposed, e.g. in prEN 197

Portland cements	(CE I)
Portland composite cements	(CE II)
blast furnace cements	(CE III)
pozzolanic cements	(CE IV)

In all cases, for the purposes of calculation, cement is understood to exclude calcium sulphate and additives.

As a guideline the following limits for the composition of the various types of cement may be given which are identical to those presented in prEN 197. Portland cement should have a clinker content of at least 95%. Portland composite cement should consist of at least 65–80% Portland cement clinker and up to 35% ground granulated blast furnace slag, up to 28% natural or artificial pozzolans and up to 20% ground limestone filler. Blast furnace cements should have a minimum of 20% of Portland cement clinker and up to 80% of ground granulated blast furnace slag. Pozzolanic cements should have a minimum of 60% of Portland cement clinker and up to 40% of natural or artificial pozzolans.

In addition, cements may differ with respect to their strength class as well as with respect to their rate of strength development. Also cements with special properties may be used for special applications such as cements with a low heat of hydration, sulphate resistant cements or cements with a low effective alkali content.

* Since the approval of this Model Code by the CEB General Assembly a CEN standard for cement ENV 197 has been approved by the CEN member countries. The types and strength classes as well as the composition of cements given in ENV 197 differ from those referred to in this Model Code.

As an example in pr EN 197 distinction is made between the four types of cement listed above (CE I to IV), between 2 strength classes (32.5 and 42.5 MPa) and between normal and rapid hardening cements (R). Many national standards also specify a third strength class around 52.5 MPa, which is referred to in subsequent sections.

In this Model Code reference is made to cements in more general terms

slowly hardening cements	(SL)	e.g. CE 32.5
normal hardening cements	(N)	e.g. CE 32.5 R; CE 42.5
rapid hardening cements	(R)	e.g. CE 42.5 R
rapid hardening high strength cements	(RS)	e.g. CE 52.5

National or international standards may contain restrictions concerning the type of cement to be used for certain applications and additional requirements for the minimum strength and composition of cements used for pretensioned prestressed concrete or for cement grout used for prestressing tendons.

d.4.2.2. Handling and storage

Cement should be protected from moisture and impurities during transportation and storage. Cement stored in the open absorbs moisture and carbon dioxide from the air, which causes the cement to cake and impair its hydration capacity. Finely ground, rapid hardening high strength cements are particularly sensitive in this respect. Except in very dry climates paper bags do not provide sufficient protection for prolonged storage even under cover. Special packaging measures are required for cement expected to be stored for prolonged periods or at a high relative humidity. Before using cement, the consumer should make sure that it still complies with the applicable standard specifications.

The various types of cement should be clearly marked and so stored that the wrong type cannot be used by error.

d.4.3. Aggregates

d.4.3.1. Types

The aggregates used for concretes covered by this Model Code can be natural or artificial mineral substances, either crushed or uncrushed and with particle sizes, grading and shapes such that they are suitable for the production of concrete. The individual particles generally have a range of sizes and have a dense or a porous structure. Further definitions are given in section d.3.

Aggregates can be further distinguished according to their rock and mineral type, their constituent materials, or according to some properties of practical importance, e.g. the shape and surface texture of the particles and their strength and durability.

d.4.3.2. General requirements

Normally, aggregates shall satisfy the requirements in the national standards for the particular applications envisaged. In special cases, it is possible to use aggregates which do not comply with the standards or aggregates for which the available standards are not applicable, provided that there are sufficient data or experience to show that concrete made with these materials is satisfactory and that the suitability of the aggregate for the particular conditions of use has been checked.

It is a general requirement that aggregates shall not become soft and shall not be excessively friable or expand excessively when wetted. They shall not be liable to decomposition, shall not combine with the products of hydration of the cement to form compounds which damage the concrete, e.g. alkali-silica reaction, and shall not cause corrosion of embedded steel reinforcement.

In order to satisfy these requirements the amount of harmful substances in the aggregates should be limited. Harmful in particular are clay and silt, i.e. particles with a diameter less than about 0.06 mm; organic compounds such as humus, coal or wood; sulphates and salts which may be aggressive to embedded steel such as nitrates, halogenides, in particular chlorides. Some standards limit the amount of silt in sand to 3 or 4 percent by weight and in coarse aggregates to 1 to 2 percent by weight. Sulphates, expressed as SO_3, often are limited to 1 percent by weight. With regard to the limitation of chlorides refer to clause d.6.3.5. Reactive aggregates are dealt with in clause d.6.3.3.3.

Depending on their application, the aggregates shall also satisfy certain requirements as to grading, purity, strength, shape of the particles, surface texture and resistance to frost and abrasion. In the case of lightweight aggregates, the porosity and water absorption as a function of time are also of special interest (refer to section d.16).

For the design and construction, account shall be taken of any unusual effects which the aggregates might be known to have on certain properties of the concrete, e.g. on its strength, density, shrinkage, moisture movement, thermal expansion, elastic modulus or durability. For example, some dolerites and sandstones shrink when drying; concrete made with these can shrink significantly more than estimated from section 2.1 of this Model Code.

d.4.3.3. Aggregates from marine sources

Aggregates from marine sources may be used provided that their chloride content complies with national standards or the information given in clause d.6.3.5, and that they satisfy all other requirements for the particular application of the aggregates.

Large quantities of hollow or flat shells in marine aggregates can have an adverse effect on the workability of the fresh concrete and on the properties of the hardened concrete.

d.4.3.4. Handling and storage

The aggregate must not become contaminated with other materials during transport or storage. If aggregates of different gradings or of different types are delivered separately, they should not be mixed inadvertently. Segregation of the different sizes of aggregates within a size range should be prevented.

d.4.4. Mixing water

Water to be used as mixing water should comply with national standards and shall not contain harmful ingredients in such quantities as to affect the properties of the fresh or hardened concrete or to reduce the protection of the reinforcement against corrosion.

In case of doubt, the water should be tested beforehand. Water containing oil, fat, sugar or a high amount of humic acids is unsuitable. Seawater shall not be used for the production of prestressed concrete and in general of reinforced concrete. Its use for reinforced concrete is permissible in some countries in exceptional cases subject to restrictions in the choice of cements and the total chloride content of the concrete. In such cases only a part of the total mixing water should be seawater.

Water previously used to remove remnants of concrete from installations of ready-mixed concrete plants and from transport vehicles may be recycled and used as mixing water subject to a number of precautions.

- Recycling water has to satisfy the general requirements for mixing water of concrete.
- Special measures are necessary if part of the recycling water has been used to clean machinery or the outside of transport vehicles so that the water is contaminated with grease or oil.

- Silt or fine aggregate particles in the water should either be distributed uniformly or separated from the water by sedimentation.
- The content of silt or fine aggregate particles should be determined on the basis of density measurements of the water, and has to be taken into account in mix design.
- Recycling water should not be used for air-entrained concrete or for concrete with a high sulphate resistance unless it is shown that the recycling water does not contain too high amounts of C_3A. Limits for allowable C_3A content as given in national standards should be followed.
- Recycling water should be used with caution for architectural concrete since its use may lead to a non-uniform appearance of the concrete surfaces.

d.4.5. Admixtures and additions

d.4.5.1. Admixtures

Admixtures as defined in section d.3 are added to the concrete to improve certain properties of the concrete by their chemical and/or physical effects. In general, they are added in small quantities less than approx. 50 g or 50 cm^3/kg of cement. Depending on the amount and composition of an admixture its water content may have to be taken into account in calculating the water/cement ratio. In special cases cements containing admixtures may be used instead of adding admixtures to the mix.

Types of admixtures include plasticizing water reducing admixtures, high range water reducing superplasticizing admixtures, water retaining admixtures, air entraining admixtures, set accelerating admixtures, hardening accelerating admixtures, set retarding admixtures and water repellent admixtures. Whereas the efficacy of these admixtures is well established and accepted it is still controversial for corrosion inhibiting admixtures.

The user should have all necessary information on the constituents and properties of the admixtures and their effect on concrete and reinforcement including information on the typical dosage, the detrimental effects of under-dosage and over-dosage, whether or not the admixture contains substances enhancing steel corrosion such as chlorides and, if so, the chloride content and the permissible duration and conditions of storage.

Admixtures should be transported and stored so that their quality is not affected by physical and chemical influences such as frost or high temperatures. They should be clearly marked and so stored that error is excluded.

The admixtures shall not adversely affect those properties of the concrete which are important for a particular application. They shall neither impair the durability of the concrete nor combine with the ingredients to form harmful compounds or endanger the protection of the reinforcement against corrosion. Chlorides, in particular, may increase the risk of corrosion. For this reason, the use of admixtures containing chlorides for reinforced concrete or for concrete which might come into contact with reinforced concrete is prohibited outright in some countries and strongly restricted in others. With regard to the total chloride content of a concrete mix refer to clause d.6.3.5.

Whilst improving certain properties of the concrete, an admixture can adversely affect others, and it is therefore necessary to carry out tests to verify the suitability of the admixture. These tests should also provide data on the quantity of admixture to be added in an individual case.

d.4.5.2. Additions

Additions as defined in section d.3 are added to the concrete in order to improve certain properties or to achieve special properties. Because of their larger amounts, the volume of additions always has to be taken into account in mix design.

Inert fines, pigments etc. are classified as Type I additions. Type II additions include latent hydraulic or pozzolanic substances such as granulated blast furnace slag, pulverized fuel ash or silica fume. Additions should comply with national standards, and their effectiveness should be checked by trial mixes.

Additions must not be harmful and shall be compatible with the other ingredients of the concrete. This means, for example, that their contents of chloride, sulphur or magnesium and their loss on ignition must not exceed certain limiting values; otherwise they can exert a harmful effect (e.g. on the durability of the concrete or on its ability to protect the reinforcement against corrosion) which may not be detected by short-term tests. The user should have the necessary information regarding the nature of the additions and their permissible content in concrete.

d.4.5.3. Handling and storage

The transport and storage of admixtures and additions should be so arranged that their quality is not affected by physical and chemical influences. The packing materials and/or the delivery papers should clearly indicate the type of admixture or addition concerned, together with the conditions of storage and use.

d.5. Classification of concrete

In design concrete is generally classified on the basis of its compressive strength. For particular applications it may also be classified on the basis of its density or potential durability.

d.5.1. Classification by strength

Concrete may be classified on the basis of its characteristic compressive strength f_{ck} as given in subsection 2.1.1 of this Model Code.

The characteristic strength is defined as that value of strength below which 5% of the population of all possible strength measurements of the specified concrete are expected to fall. This code is based on the uniaxial compressive strength of cyclinders, diameter 150 mm, height 300 mm, stored in water at 20 $\pm$ 2°C until tested at an age of 28 days in accordance with ISO 1920, ISO 2736/2 and ISO 4012. Where specimens other than cylinders 150/300 mm are used and where they are stored in other than the standard environment conversion factors for the compressive strength of specimens of different size, shape and storage conditions should be determined experimentally or according to rules given in national standards. Clause 2.1.3.2 of this Model Code gives such conversion factors for cubes 150/150/150 mm made of normal weight concrete.

For special cases, it may be necessary to define a minimum strength limit at an earlier age in addition to the governing 28-day strength, e.g. to enable the forms to be struck earlier, for earlier prestressing or for earlier transportation of precast elements. In special cases it is also permissible for the required strength to be attained at a later age, e.g. where slowly hardening cements are used, provided that the expected loading history as well as the durability of the structure are taken into consideration.

d.5.2. Classification by density

Concrete may be classified on the basis of its density as defined in section d.3 (7), (8) and (9)

- normal weight concrete having an oven-dry density greater than 2000 kg/m^3 but not exceeding 2800 kg/m^3
- lightweight aggregate concrete having an oven-dry density less than 2000 kg/m^3
- heavyweight concrete having an oven-dry density larger than 2800 kg/m^3.

The treatment (placement, compaction, etc.) of heavyweight concrete demands observance of additional rules not given in this appendix.

d.5.3. Classification by durability

The durability of concrete is understood to be its resistance to physical and chemical attack such as frost or elevated temperatures, carbonation, sulphate attack etc. The resistance of concrete to such actions is governed primarily by its resistance to the ingress of aggressive media and thus by the capillary porosity of the hydrated cement paste as well as by entrapped air. A dense paste with a low capillary porosity is in most instances more durable than a paste with a high capillary porosity and a coarser pore system.

There is no generally accepted method to characterize the pore structure of concrete and to relate it to its durability. However, several experimental investigations have indicated that concrete permeability both with respect to air and to water is an excellent measure for the resistance of concrete against the ingress of aggressive media in the gaseous or in the liquid state and thus is a measure of the potential durability of a particular concrete.

There are at present no generally accepted methods for a rapid determination of concrete permeability and of limiting values for the permeability of concrete exposed to different environmental conditions. However, it is likely that such methods will become available in the future allowing the classification of concrete on the basis of its permeability. Requirements for concrete permeability may then be postulated; they would depend on exposure classes i.e. environmental conditions to which the structure is exposed.

Though concrete of a higher strength class is in most instances more durable than concrete of a lower strength class, compressive strength per se is not a complete measure of concrete durability, because durability primarily depends on the properties of the surface layers of a concrete member which have only a limited effect on concrete compressive strength.

d.6. Concrete performance requirements

d.6.1. General considerations

The composition of a particular concrete should be so chosen that a required performance both with regard to strength and to durability is assured. However, not all concrete performance requirements can be evaluated sufficiently on the basis of direct experiments. Therefore, until adequate test methods are developed, the performance requirements—particularly those for concrete durability—have to be expressed on the basis of certain rules with regard to concrete composition and choice of materials.

Consequently, the components of the concrete, cement, aggregate, water, admixtures and additions must be selected and the mix proportions so chosen that all relevant performance criteria are met. In particular con-

sistence and resistance to bleeding of the fresh concrete, density, strength and other mechanical properties of the hardened concrete as well as its durability and ability to protect embedded steel against corrosion are of importance.

d.6.2. Requirements for strength

The compressive strength of concrete depends primarily on the water/cement ratio, on the degree of hydration and, therefore, on age and curing of the concrete as well as on the strength class of the cement used. It is also influenced by type and strength of the aggregates as well as by type and amount of additions.

The relations between water/cement ratio and concrete compressive strength are not unique. Therefore, they have to be determined at least for each combination of type of cement, type of aggregate and for a given concrete age. As a rough guideline the relations between water/cement ratio and characteristic compressive strength f_{ck} at an age of 28 days for concretes made with cements of different strength classes are given in Fig. d.1. Relations between concrete compressive strength and other mechanical properties are given in section 2.1 of this Model Code. With regard to the effect of additions refer to clause d.6.3.3.2.

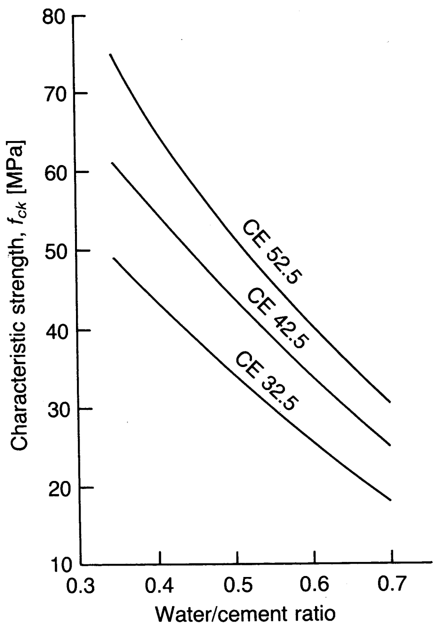


Fig. d.1. Approximate relations between the water/cement ratio and the characteristic compressive strength f_{ck} of concrete for various strength classes of cement and quartzitic aggregates

d.6.3. Requirements for durability

d.6.3.1. General considerations

To produce a durable concrete, which will withstand exposure to certain environmental conditions as given in clause d.6.3.2 and which protects the reinforcing steel against corrosion during the intended lifetime, the following aspects should be taken into account

- choice of suitable constituent materials containing no ingredients harmful to the durability of the concrete or causing corrosion of the steel
- choice of a concrete composition such that the concrete
  - — satisfies all specified performance criteria for fresh and hardened concrete
  - — can be placed and compacted to form a dense cover to the reinforcement
  - — avoids harmful internal actions, e.g. alkali-silica reactions
  - — withstands external actions, e.g. environmental influences such as weathering, gases, liquids and soil
  - — withstands mechanical attacks, e.g. abrasion
- mixing, placing and compacting of the fresh concrete such that the concrete constituents are distributed uniformly in the mix and are not segregated and that the concrete achieves a closed structure
- curing of the concrete such that especially the surface zone and the cover of the reinforcement obtain the desired properties to be expected from the mix.

d.6.3.2. Exposure conditions

The requirements to be met by the concrete depend on the environment to which the concrete is exposed. Environment in this context implies chemical or physical actions resulting in effects which are not considered as loads in structural design.

In many instances different parts of a structure may be subjected to different exposure conditions. In some cases local conditions, i.e. the 'micro-climate' may be decisive for the durability of the entire structural member. The major environmental conditions for concrete are classified according to Table 1.5.1 in subsection 1.5.2 of this Model Code as well as in Table d.1 of this appendix. When selecting an exposure class for a particular application from Table d.1 it should be borne in mind that for some elements more severe conditions may prevail during construction than assumed for the completed structure.

The exposure conditions given in Table d.1 apply to concrete. A somewhat different classification is necessary to describe the environmental conditions aggressive to the reinforcement (refer to 'Durable Concrete Structures—CEB Design Guide').

d.6.3.3. Choice of materials

1. Types and strength classes of cement

Already at the design stage of a concrete structure the following aspects, which are relevant for the choice of type and strength class of the cement to be used should be kept in mind:

- required characteristic strength of the concrete
- required rate of strength development
- dimensions of the structure including cover to reinforcement, with regard to the rate of hydration
- environmental conditions during casting
- exposure conditions of the finished structure, in particular aggressive environments such as sulphate attack, exposure to freezing and thawing with or without de-icing chemicals
- curing of the concrete.

If the required characteristic strength of the concrete is high, the use of cements of a high strength class may be advantageous. Other parameters being equal, Portland cements (CE I according to clause d.4.2.1), other

Table d.1. Exposure classes for concrete related to environmental conditions (from ENV 206)

Exposure class	Environmental conditions
1. Dry environment	E.g. • interior of buildings used for normal habitation or offices
2. Humid environment (*a*) Without frost	E.g. • interior of buildings where humidity is high* • exterior components • components in non-aggressive soil and/or water
(*b*) With frost	E.g. • exterior components exposed to frost • components in non-aggressive soil and/or water and exposed to frost • interior components when the humidity is high and exposed to frost
3. Humid environment with frost and de-icing agents	E.g. • interior and exterior components exposed to frost and de-icing agents
4. Seawater environment (*a*) Without frost	E.g. • components completely or partially immersed in seawater or in the splash zone • components in saturated salt air (coastal area)
(*b*) With frost	E.g. • components partially immersed in seawater or in the splash zone and exposed to frost • components in saturated salt air and exposed to frost
5. Aggressive chemical environment† (*a*)	Slightly aggressive chemical environment (gas, liquid or solid) Aggressive industrial atmosphere
(*b*)	Moderately aggressive chemical environment (gas, liquid or solid)
(*c*)	Highly aggressive chemical environment (gas, liquid or solid)

* E.g. in commercial laundries.
† Chemically aggressive environments are classified further in clause d.6.6.4, Table d.3. They may occur alone or in combination with exposure classes 1–4.

types of cement with a high content of Portland cement clinker and in particular R-cements as defined in subsection d.4.2 exhibit a strength development which is more rapid than that of cements having a higher content of additional constituents. Therefore, they are especially suitable where a concrete strength at a concrete age less than 28 days is specified.

Where the dimensions of a concrete member are large, cements generating low amounts of heat of hydration should be used in order to avoid thermal cracking. Many slowly hardening cements containing higher contents of constituents other than Portland cement (CE II, CE IV according to clause d.4.2.1) satisfy this requirement.

Low ambient temperatures during casting may require rapid hardening cements whereas low heat cements are advantageous where the ambient temperatures during casting are high.

Special cements may be required for certain exposure conditions such as

sulphate resistant cements if the concrete is exposed to water with a sulphate content exceeding approx. 600 mg/kg or to soil with a sulphate content exceeding approx. 3000 mg/kg. Portland cements with a C_3A content less than approx. 3% by mass as well as blast furnace slag cements with a slag content > 65% are generally considered sulphate resistant. Also some Portland composite cements may have a high resistance to sulphates even if their C_3A content is higher than 3%. Although seawater may contain a substantial amount of sulphates, the use of sulphate resistant cements is not mandatory for concrete exposed to seawater.

In cases where the use of reactive aggregates cannot be avoided, special cements with a low content of Na_2O and K_2O result in more durable concrete.

The effectiveness of air-entraining agents in protecting concrete against freezing, thawing and de-icing agents depends to some extent on the type of cement. For reinforced concrete members exposed to de-icing agents which contain chlorides, the resistance of the concrete to the penetration of chlorides is of significance which may be influenced also by the type of cement. Concrete made of blast furnace cements (CE III) may have a particularly high resistance to the penetration of chlorides; however, for high slag contents their scaling resistance may be reduced.

It is also worth noting that, other parameters being equal, i.e. water/cement ratio, cement content and curing, concretes made with cements having substantial amounts of constituents other than Portland cement (CE II–CE IV) carbonate at a higher rate than concretes made of Portland cement (CE I). This effect can be offset by the use of lower water/cement ratios or more intensive curing.

Depending on the type and strength class of the cement more or less intensive curing of the concrete may be required. Many cements containing larger amounts of constituents other than Portland cement clinker (CE II–CE IV) are generally more sensitive to curing than most Portland cements of the same strength class. However, after prolonged curing or in a humid climate their pore structure is particularly dense and impermeable resulting in concretes of high durability (also refer to section d.12).

For these reasons not all types of cement should be used for all conditions to which a concrete member may be exposed. National standards which are based on local experiences with certain types of cement should, therefore, be followed. This is particularly true for cements used for prestressed concrete structures.

2. Additions

Most additions with hydraulic properties, Type II according to d.3 (11), may influence the durability of concrete in a way similar to that of additional constituents of cements as described in the preceding sections.

National standards or regulations may permit consideration of some additions as part of the total cement content and taking them into account when calculating an effective water/cement ratio $(w/c)_{ef}$

$$(w/c)_{ef} = \frac{w}{c + \alpha f}$$

where

w is the water content
c is the cement content
f is the content of addition
α is the efficiency coefficient.

The efficiency coefficient α is not a unique value even for a given type of addition. It may depend on concrete age, amount of the addition f, type of

cement, the particular combination of addition and cement, and may be different when considering the effect of water/cement ratio on strength or on durability.

For fly ash obtained from the flue gases of furnaces fired with pulverized anthracite or bituminous coal, values of α may range from $\alpha \cong 0$ to $\alpha \cong 0.5$ for durability considerations. As long as the ratio of fly ash content to cement content $f/c < 0.15$ $\alpha \cong 0.4$ may apply for strength considerations. For some silica fumes values of α up to 0.7 may be valid.

The total amount of pozzolanic additions has to be limited since, particularly at higher ages, such additions may react with the calcium hydroxide formed by the hydration of Portland cement clinker. This may impair the alkalinity of the concrete and thus its ability to protect embedded steel against corrosion. Where fly ash or natural pozzolans are used, the content of Portland cement clinker given as a percentage of the sum of cement and additions should not be less than 60%. For cements containing Portland cement clinker and ground granulated blast furnace slag as the only main constituents, the content of clinker should not be less than 20%.

The content of silica fume should not exceed 15% of the content of cement.

Currently, CEN-Standards are under preparation in which requirements to be satisfied by additions are specified.

3. Aggregates

Aggregates should satisfy the rules set out in subsection d.4.3.

Where the use of reactive aggregates is inevitable, one or a combination of the following precautions should be taken

- limit the total alkali content of the concrete mix
- use a cement with a low content of effective alkali
- limit the degree of saturation of the concrete, e.g. by impermeable membranes.

For further details the requirements of national standards or regulations should be followed taking account of previous long term experience existing with a particular combination of cement and aggregate.

d.6.3.4. Cement content and water/cement ratio

So long as concrete cannot be characterized in terms of durability classes as indicated in subsection d.5.3, durability of a particular concrete may be related to its strength class or its composition, in particular type of cement, minimum cement content and maximum water/cement ratio. In addition, minimum strength classes may be required for a given exposure class.

To ensure a high resistance to the penetration of aggressive substances, the water/cement ratio should be the lower, the more severe the exposure of the concrete member. A minimum cement content should be maintained in order to guarantee the alkalinity of the concrete required for corrosion protection of embedded steel reinforcement and to ensure workability of the fresh concrete for a given water/cement ratio. Recommended values of minimum cement content and maximum water/cement ratios for different exposure classes together with other requirements are given in Table d.2.

The values for minimum cement content given in Table d.2 apply for concrete with a maximum aggregate size of 32 mm. The amount of cement paste required for a given workability of the fresh concrete, generally decreases as the maximum aggregate size increases. Therefore, for concretes with a maximum aggregate size larger than 32 mm, values of the minimum

Table d.2. Durability requirements related to environmental exposure (based on ENV 206, Table 3)

	Exposure class according to Table d.1								
	1	2a	2b	3	4a	4b	5a	5b	5c†
Max. w/c ratio* for									
plain concrete	–	0.70	–	–	–	–	–	–	–
reinforced concrete	0.65	0.60	0.55	0.50	0.55	0.50	0.55	0.50	0.45
prestressed concrete	0.60	0.60	–	–	–	–	–	–	–
Min. cement content* in kg/m³ for									
plain concrete	150	200	200	–	–	–	200	–	–
reinforced concrete	260	280	280	300	300	300	280	300	300
prestressed concrete	300	300	300	–	–	–	300	–	–
Min. air content of fresh concrete in % for nominal max. aggregate size of:‡									
32 mm	–	–	4§	4§	–	4§	–	–	–
16 mm	–	–	5§	5§	–	5§	–	–	–
8 mm	–	–	6§	6§	–	6§	–	–	–
Frost resistant aggregates	–	–	Yes	Yes	–	Yes	–	–	–
Impermeable concrete according to d.6.6.1	–	–	Yes	Yes	Yes	Yes	Yes	Yes	Yes

* Max w/c may be replaced by $(w/c)_{ef}$ and min. cement content may be replaced by $(c + \alpha f)$ where national standards or regulations allow. Refer to clause d.6.3.3.2.
† In addition, the concrete should be protected against direct contact with the aggressive media, e.g. by coatings.
‡ With a spacing factor of the entrained air void system $\leqslant 0.20$ mm.
§ In cases where the degree of saturation is high for prolonged periods of time, e.g. for structures in the tidal zone, parts of locks, etc.

cement content lower than those given in Table d.2 apply, whereas higher values are required for concretes with a maximum aggregate size smaller than 32 mm.

The values for maximum water/cement ratio, w/c, and minimum cement content, c, given in Table d.2 are mean values. According to ENV 206 for maximum w/c conformity is assumed if the mean value of w/c is not higher than the values given in Table d.2, and if single values do not exceed these values by 0.02. For minimum c conformity is assumed if the mean value of c is not less than the values given in Table d.2 and if single values are not lower than 5% by weight of the specified value.

The durability requirements laid down in Table d.2 apply to conventional concretes. For some prefabricated members or for high strength concretes, deviations from these requirements may be justified; e.g. for high strength concrete to be used within exposure classes normally requiring entrained air this provision may not be necessary, provided adequate frost resistance of the particular concrete is documented by experiments.

d.6.3.5. Maximum permissible quantity of deleterious substances in the concrete

The durability of the concrete and the protection afforded to embedded reinforcement can be adversely affected by any excess of deleterious substances in the components of the mix. Generally applicable limits cannot be quoted, because the permissible amount of harmful substances depends on the particular use of the concrete, on the ambient conditions to which the concrete is exposed and on the type and composition of all concrete constituents.

Excessive amounts of sulphate and free magnesium oxide in the constituent materials can lead to deterioration by expansion cracks. Free chloride ions Cl^- in the concrete may be particularly harmful in relation to corrosion protection of embedded reinforcement. Experience has shown that a content of free chlorides exceeding 0.35 to 1.0% of the weight of cement in the concrete may cause corrosion. Even lower chloride contents may be harmful, e.g. in carbonated concrete or for prestressing strands. Therefore, expert opinion should be obtained in cases where the acceptable limit of chlorides is in doubt. Also refer to 'Durable Concrete Structures —CEB Design Guide'.

Also national and international standards provide information; e.g. ENV 206 gives limits on the Cl^- content by weight of cement, determined in accordance with EN 196, Part 21, of 1.0% for plain concrete, 0.4% for reinforced concrete and 0.2% for prestressed concrete.

d.6.4. Requirements for workability of fresh concrete

Workability of the fresh concrete i.e., its coherence and its placing, compacting and finishing properties may be expressed in terms of its consistence. The level of consistence must be such that the fresh concrete is easily workable without becoming segregated and that it can be fully compacted with the equipment available.

The consistence required for a particular application depends on the size of the structural member, the presence and spacing of reinforcement, the available equipment for compaction and the environmental conditions.

The consistence of the fresh concrete depends on the water content, the fineness and the quantity of the fines, and on the grading and the nature of the aggregates. It can be influenced by certain admixtures and additions.

To ensure proper compaction of concrete cast in situ, it is recommended that the consistence of the concrete at the time of placing is high, e.g. classes S3 or F3 as defined in clause d.7.1.1 unless other measures are taken.

For special applications, such as for some production procedures of precast concrete elements, very dry concrete mixes–zero-slump concretes —not covered by the consistency ranges given in clause d.7.1.1 may be particularly suitable.

Attention should be paid to a possible reduction of consistence of the fresh concrete soon after mixing. It may occur in a dry environment and at high ambient temperatures or when certain cements or admixtures, such as high range water reducing superplasticizing admixtures, are used.

The consistence may be measured by means of various standard tests which are given in clause d.7.1.1.

d.6.5. Mix design

d.6.5.1. Grading of aggregates

1. General considerations

The aggregate for concrete consists of a mixture of particles of different sizes which are combined in accordance with certain requirements. In order to achieve the required or prescribed grading and to keep it within acceptable limits, the aggregate in the concrete should be composed from materials having a narrower range of particle sizes.

The grading of individual materials and of the combined aggregate is established with test sieves on the basis of national or international standards. ISO 565 proposes the following sieve sizes: 0.125, 0.25, 0.5, 1.0, 2, 4, 8, 16, 31.5 and 63 mm.

The principal aspects concerning overall aggregate gradings are as follows.

(*a*) Optimum cement content and low water demand. In this respect, it is advantageous and useful to have aggregate gradings with a relatively low sand content, a high proportion of coarse particles and a small amount of interstitial voids.

(*b*) The concrete should not become segregated during handling, placing or compaction, should be sufficiently workable to be compacted with the available equipment and should attain a close textured surface after finishing. To meet this requirement, an optimum content of fines is needed, this requirement being partly in conflict with clause (*a*). The optimum grading will depend on the handling, placing, compacting and finishing conditions, on the type of structure and on the properties of the aggregates to be used (e.g. particle shape, surface texture and available gradings).

(*c*) The maximum particle size depends on the dimensions of the concrete member, the thickness of the concrete cover, the spacing of the reinforcing bars, the handling and placing conditions, and sometimes, in the case of lightweight aggregate concrete, on the strength class. It should not exceed one-third of the smallest dimension of the member and should generally be less than the spacing of the bars and their distance from the shuttering. It is recommended to use a nominal maximum particle size which corresponds to one of the mesh sizes specified in the applicable national or international standard.

The grading of aggregates can be represented by grading curves. It may also be sufficient to specify only the ratio of fine to coarse aggregate. In this case, the grading of the fine and the coarse aggregate should comply with certain requirements. In all cases, the essential criterion for the composition of the aggregate mix is the absolute volume and not the weight of the different grading groups.

An aggregate may be continuously graded or gap graded. A continuously graded aggregate includes all particle sizes from the finest to the largest. A gap graded aggregate is one in which one or more of the intermediate grading groups are missing.

2. Continuous grading curves

Figures d.2 to d.5 show examples of grading curves. For natural sand and gravel gradings in zone 3 are preferred although gradings in zone 4 are also acceptable. Aggregates in zone 1 tend to produce harsh concretes which are difficult to work and liable to bleed: these difficulties can be minimized by blending the aggregate with other aggregates, if available, or by the use of air entrainment or additions (refer to clause d.6.5.1.4).

Aggregates in zone 5 may require large amounts of water and in consequence high cement contents for a given water/cement ratio to achieve a

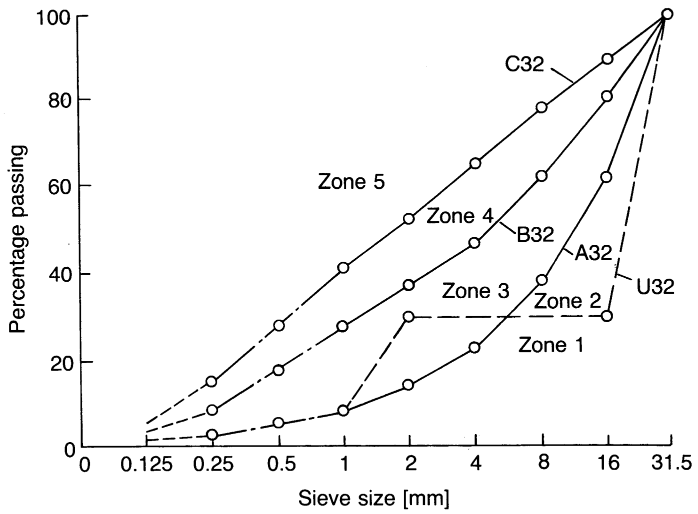


Fig. d.2. Examples of grading curves for a maximum aggregate particle size of 8 mm

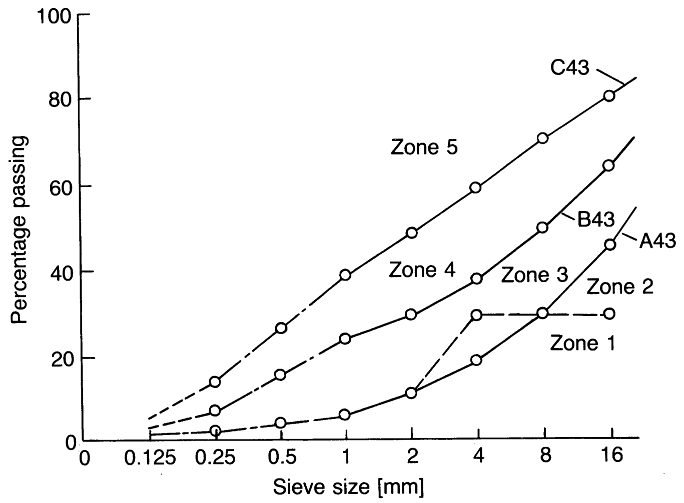


Fig. d.3. Examples of grading curves for a maximum aggregate particle size of 16 mm

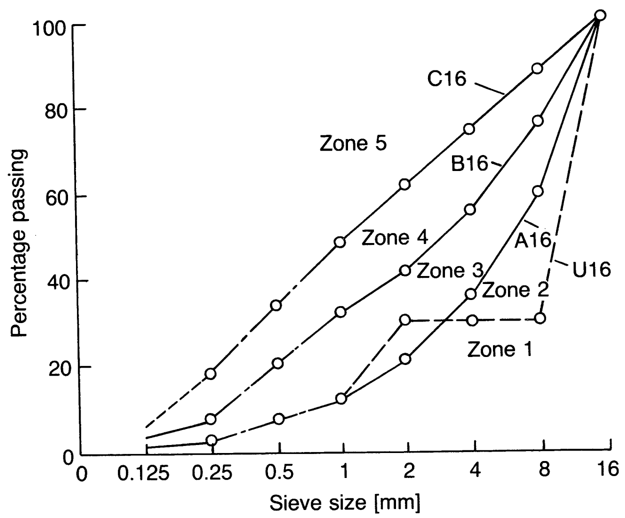


Fig. d.4. Examples of grading curves for a maximum aggregate size of 32 mm

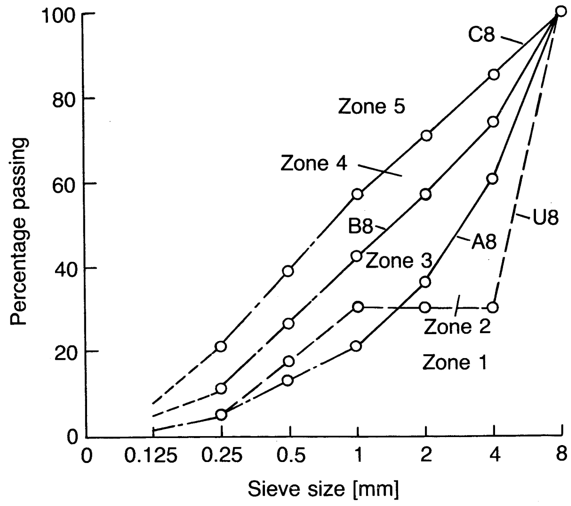


Fig. d.5. Examples of grading curves for a maximum aggregate size of 63 mm

given strength. This can be modified by blending the aggregates with other aggregates if available.

Slightly different rules may have to be applied for crushed sand and crushed coarse aggregates.

3. Gap graded mixes

Gap graded aggregates with grading curves between the grading curves (U) and (C) in Figs. d.2 to d.5 are generally suitable.

4. Content of ultrafine material

Ultrafine material comprises all components having particle sizes smaller than 0.25 mm including the cement and the very fine particles in the aggregate and in the additions, if any. Particles smaller than 0.125 mm in size are particularly effective.

In order that concrete may be cohesive, making it easier to work, it should contain a sufficient quantity of ultrafine material. This is especially important for concrete which is to be transported over long distances or conveyed through pipes, for concrete in thin walled structural components with closely spaced reinforcing bars and for concrete having a high resistance to water penetration (refer to clause d.6.6.1).

If the cement content is low and if the sand contains only small amounts of ultrafine particles very fine grained mineral substances, non-deleterious for concrete may be added. To verify that the ultrafine particles do not contain harmful amounts of fine substances such as clays, organic material or iron hydroxides they should be tested according to national standards, e.g. the methylene blue test or the sand equivalent test. Entrained air tends to reduce the necessary amount of ultrafine material. As a guidance it may be assumed that approx. 10 cm^3 of entrained air replace approx. 5 cm^3 by absolute volume of ultrafines.

The optimum amount of ultrafines depends on the grading of the aggregates. However, the following aspects should be taken into account.

- Too high an amount of ultrafines will reduce the workability of the fresh concrete unless water reducing or high range water reducing superplasticizing admixtures are used.
- A replacement of cement by ultrafines beyond the limits given in clause 6.3.3.2 can adversely affect concrete durability.
- If the addition of ultrafines allows a reduction of cement content without an increase of the water/cement ratio, shrinkage and creep of the concrete are reduced.

d.6.5.2. Cement content and water/cement ratio

In many instances the water/cement ratio of a mix is governed by the strength requirements as indicated in subsection d.6.2. Even if the strength requirement would allow a higher water/cement ratio it should not exceed the values given in Table d.2 for different exposure conditions.

The cement content is controlled by the water demand of the aggregates to achieve a certain consistence of the fresh concrete together with the water/cement ratio. Where no previous experience for the water demand of a given type and grading of aggregates is available it has to be determined by means of trial mixes. In no case should the cement content fall below the minimum values given in Table d.2, taking into account the first footnote.

d.6.5.3. Admixtures and additions

Admixtures and additions should only be added to the mix in such quantities that they do not reduce the durability of the concrete and do not cause corrosion of the reinforcement.

Therefore, as a guide the total amount of admixtures, if any, should not exceed 50 g/kg or 50 cm^3/kg cement. On the other hand, the amount of each admixture added should not be less than 2 g/kg cement in the mix unless it is dispersed in part of the mixing water prior to adding it to the mix. This is necessary to ensure uniform distribution of the admixtures within the mix.

Liquid admixtures in quantities exceeding 3 dm^3/m^3 of concrete should be taken into account when calculating the water/cement ratio.

A certified product declaration based on tests and including instructions for the admixture used should be available at the place where the concrete is produced.

The total amount of additions, particularly of highly reactive pozzolans should be limited as indicated in clause d.6.3.3.2.

d.6.6. Concrete with special properties

The general requirements regarding concrete composition to meet both strength and durability requirements have been dealt with in subsections d.6.2 and d.6.3. In the subsequent paragraphs additional information is given on the production and composition of concrete subjected to special service conditions. In this context also refer to the information given in 'Durable Concrete Structures—CEB Design Guide'.

d.6.6.1. Concrete with high resistance to water penetration

The resistance of concrete to water penetration may be tested e.g. in accordance with ISO 7031 (refer to clause d.7.2.5).

The resistance of fully compacted concrete against penetration and saturation by water depends primarily on the porosity of the hardened cement paste. The water/cement ratio should, therefore, be low and should not exceed 0.55 in structural components with a thickness of 100–400 mm or 0.60 in thicker components. In addition, careful curing is required.

d.6.6.2. Concrete with high resistance to freezing and thawing

Concrete which is critically saturated will be damaged by frequent cycles of freezing and thawing. Such damage can be prevented by limiting the likelihood of critical saturation i.e. low water/cement ratio and sufficient curing, and by the use of air-entraining agents.

In Table d.2 minimum values of the air content of fresh concrete are given for concretes made of aggregates with different maximum sizes. The effectiveness of an air-entraining agent is characterized by the spacing-factor, i.e. the largest distance of a point within the cement paste from an air void and by the microporosity of the cement paste. The spacing factor should

not exceed 0.20 mm. The microporosity, which may be defined as the content of pores smaller than 300 μm, should not be less than 1.5%. In addition, the frost resistance of the aggregates should be checked by suitable tests.

d.6.6.3. Concrete with high resistance to freezing and thawing and de-icing chemicals

The effects of freezing and thawing of concrete generally become more severe under the simultaneous action of freezing and thawing and de-icing chemicals.

Under such conditions the use of air-entraining agents is vital, and lower values of the water/cement ratio should be used (refer to Table d.2).

Some cements with a low percentage of Portland cement clinker have had an unsatisfactory service record in some countries when used for concretes exposed to de-icing salts unless low water/cement ratios and very good curing prevented critical saturation of the concrete. Therefore, such cements should be used cautiously under such conditions, and local experience as well as national standards and regulations should be observed.

d.6.6.4. Concrete with high resistance to chemical attack

The severity of chemical attacks depends mainly upon the nature of the aggressive substances (chemical composition and whether solid, fluid or gaseous), their pressure and their rate of flow and the ambient temperature. In the case of solids or gases the presence of moisture is of importance.

It is often difficult to judge whether a chemical attack will occur or not and to choose the necessary precautions. Therefore, in case of doubt, it is recommended that an expert opinion is obtained.

Generally, distinction can be made between three types of chemical attack

- leaching of soluble constituents from the concrete
- chemical reactions with formation of soluble compounds
- chemical reactions, where the products occupy a larger volume than the reacting constituents of the concrete, thus damaging the concrete by their expansion.

The severity of the attack of various substances in water and soils may be assessed from Tables d.3 and d.4, respectively. There, distinction is made between three degrees of severity

- slightly aggressive (exposure class 5a according to Table d.1)
- moderately aggressive (exposure class 5b according to Table d.1)
- highly aggressive (exposure class 5c according to Table d.1).

Generally, the chemical attack is more severe at higher temperatures, at higher pressure or if the concrete is exposed to mechanical abrasion due to fast flowing water or to alternating freezing and thawing. The severity can be assumed to be lower if the temperature of the water is consistently low, if the quantities of water are small or if the rate of flow of the water is slow such as in soils with a permeability coefficient $k > 10^{-5}$ m/s.

The resistance of concrete to chemical attack depends primarily on its impermeability. Therefore, the maximum water/cement ratios given in Table d.2 should not be exceeded. In addition, particularly careful curing is necessary. In cases of exposure to highly aggressive chemicals, additional protective measures such as impermeable coatings are required.

For a sulphate content exceeding 600 mg/kg in water and 3000 mg/kg in soil, the use of sulphate resistant cements is mandatory.

In soils with high contents of both sulphates and chlorides, attention should be paid to the observation that concrete made of sulphate resistant

Table d.3. Limiting values of deleterious substances in water of predominantly natural composition for the assessment of the severity of chemical attack

	Degree of severity*		
	Slight	Moderate	High
1. pH value	6.5–5.5	5.5–4.5	< 4.5
2. Carbonic acid dissolving lime (CO_2) in mg/dm^3 determined by marble test according to Heyer	15–40	40–100	> 100
3. Ammonium (NH_4^+) in mg/dm^3	15–30	30–60	> 60
4. Magnesium (Mg^{2+}) in mg/dm^3	300–1000	1000–3000	> 3000
5. Sulphate (SO_4^{2-}) in mg/dm^3	200–600	600–3000	> 3000

* The degree of severity is valid for stationary or slowly moving water under moderate climatic conditions (e.g. Central Europe). It depends upon the highest degree of severity even if it is reached only by one of the five criteria listed in the lines 1 to 5. If two or more values are found to lie in the upper quarter of a range (in the case of the pH value in the lower quarter) then the degree of severity is deemed to be raised by one step. This increase does not, however, apply to seawater.

Table d.4. Limiting values for the assessment of the deleteriousness of soils

	Degree of severity*	
	Low	Medium
1. Degree of acidity according to Baumann-Cully	> 20	–
2. Sulphate (SO_4^{2-}) in mg/kg according to Baumann–Cully	2000 to 5000	> 5000

* The degree of severity is valid for soils which are saturated at frequent intervals. It can be reduced with decreasing permeability of the soils. If the content of sulphur from sulphides exceeds 100 mg S^{2-} per kg of air-dry soil (more than 0.01% S^{2-}) or if dumps of industrial waste products are concerned, the deleteriousness should be assessed by an expert.

Portland cements generally has a lower resistance to the penetration of chloride ions, than concrete made of blast furnace slag cements with a high slag content.

In spite of the high sulphate content of seawater, the use of sulphate-resisting cement for structures exposed to seawater is not required in some countries because, provided that it is sufficiently impermeable, concrete has proved to have adequate resistance to the effects of seawater.

Apart from these general recommendations, the resistance of concrete to chemical attack and the precautionary measures to be taken should be examined in detail in every country on the basis of local experience and technology. In addition, reference is made to 'Durable Concrete Structures —CEB Design Guide'.

d.6.6.5. Concrete with high resistance to wear

Concrete exposed to highly abrasive conditions shall be adequately resistant to wear. Such an attack can occur, for example, as a result of heavy traffic, impact or sliding loads caused by poured material or of the rapid flow of water containing solids such as sand or ice.

Concrete for use under these conditions should have a strength not less than C30; high strength concrete proved to have a particularly high abrasion resistance. The fine aggregate should consist mainly of quartz or of materials of at least the same hardness; the coarse aggregate should consist of stone or of artificial material having a correspondingly high abrasion resistance. In the surface directly exposed to the abrasive forces, the aggregates shall be tightly packed, and the coarse particles must be solidly held in the mortar to prevent their being torn out. It is recommended that the aggregate should include a high proportion of coarse aggregate and have a grading which enables the grains to be closely packed with few intergranular voids (e.g. near the grading curve A or, in the case of gap graded mixes, between B and U in Figs. d.2 to d.5). The surface texture of the aggregate particles should be fairly rough and the shape of the particles should not be elongated.

At the surface of the concrete, bleeding should be minimized, and there should not be a marked laitance layer. It is, therefore, preferable to use concrete of stiff-plastic to stiff consistence or, alternatively, to undertake a special additional treatment of the concrete surface. Such measures could be applied when the concrete is in either the fresh or in the hardened state. In the fresh state, it is possible to apply vacuum treatment according to subsection d.11.4 or to generate additional compaction by power trowelling. Spraying in a dry cement-sand mix may cause surface cracking.

To ensure that the surface of the concrete is not friable, the concrete should be cured for a sufficient period i.e. at least twice as long as proposed in section d.12.

Where the abrading action is particularly severe, the provision of a special wearing surface is advisable (e.g. partial or total replacement of the aggregate by silicon carbide or carborundum).

In the hardened state, the weaker surface zone may be removed by grinding, especially for interior floors which are not exposed to frost action.

d.6.6.6. Concrete subjected to elevated temperatures

Concrete should not be exposed for prolonged periods to temperatures higher than 250°C, otherwise there may be a serious loss of strength and harmful effects on other important concrete properties. High temperatures below 250°C can have an influence on certain properties, such as creep, shrinkage or the modulus of elasticity as indicated in section 2.1 of this Model Code. Only aggregates should be used which have proved to be suitable for concrete exposed to elevated temperatures. Where concrete is exposed to high temperatures, it is important to make sure that the concrete attains a high degree of hydration, and has dried out before it is heated for the first time. Rapid changes of temperature should be avoided.

Special concretes are required in conditions of prolonged exposure to temperatures in excess of 250°C.

d.7. Verification of concrete properties

d.7.1. Fresh concrete

d.7.1.1. Consistence

The consistence of fresh concrete may be determined by means of standard tests such as the slump test (ISO 4109), the Vebe test (ISO 4110), the compaction test (ISO 4111) or the flow table test (ISO 9812). The classifications obtained from the various test methods are given in Tables d.5 to d.8.

Table d.5. Slump classes according to ISO 4109

Class	Slump in mm according to ISO 4109*
S1	10–40
S2	50–90
S3	100–150
S4	> 160

* The slump measured is to be rounded off to the nearest 10 mm.

Table d.6. Vebe classes according to ISO 4110

Class	Vebe seconds according to ISO 4110
V0	> 31
V1	30–21
V2	20–11
V3	10–5
V4	< 4

Table d.7. Compaction classes according to ISO 4111

Class	Degrees of compactability to ISO 4111
C0	> 1.46
C1	1.45–1.26
C2	1.25–1.11
C3	1.10–1.04

Table d.8. Flow classes according to ISO 9812

Class	Flow diameter in mm according to ISO 9812
F1	< 340
F2	350–410
F3	420–480
F4	490–600

The slump test is not the most suitable test for non-cohesive concrete or for concrete with a dry consistence. If the slump is less than 10 mm the test result should be recorded as less than 10 mm. It should also be stated that the concrete has a consistence stiffer than that for which the slump test is suitable.

For concrete of fluid consistence, e.g. when high range water reducing admixtures are used, the flow table test gives the most reliable results.

Since there is no generally valid relationship between the different methods of measuring consistence, the method should be agreed upon in each particular case.

d.7.1.2. Air content

The air content of the fresh concrete may be determined by the pressure method, e.g. according to ISO 4848.

d.7.1.3. Wet density

Where a determination of the wet density of the compacted fresh concrete is required it may be measured e.g. according to ISO 6276.

d.7.1.4. Water/cement ratio and cement content

Currently, no universally applicable methods are available to determine water/cement ratio and cement content of the fresh concrete on the site within a sufficiently short period of time unless additional information such as type of cement, content of ultrafines etc. is available.

The cement content has to be controlled by careful measurement of the weight of cement added to the mix. The water/cement ratio has to be calculated from the weight of water added to the mix taking into account the water content of liquid admixtures and the effective water content of the aggregates, and from the weight of cement.

Also the 28-day compressive strength may be used to verify the water/cement ratio if relations between compressive strength and water/cement ratio are available for a particular concrete mix.

d.7.2. Hardened concrete

d.7.2.1. Compressive strength

Concrete compressive strength as defined in subsection d.5.2 should be determined on cylinders, diameter 150 mm, height 300 mm, which have been stored in water at 20 ± 2°C up to the time of testing at an age of 28 days and which are tested in accordance with ISO 4012. If national standards require different types of specimens or different storage conditions up to the time of testing, conversion factors which have been verified by tests have to be applied. In clause 2.1.3.2 of this Model Code conversion factors are given which relate the strength of cubes 150/150/150 mm to the strength of cylinders 150/300 mm made of normal weight concrete. For lightweight aggregate concretes other conversion factors may be valid.

d.7.2.2. Tensile strength

The tensile strength of concrete may be determined by various methods

axial tensile strength	according to RILEM CPC 7
splitting tensile strength	according to ISO 4108
flexural tensile strength	according to ISO 4013

In clause 2.1.3.3.1 of this Model Code relations are given to estimate upper and lower limits of the tensile strength from compressive strength as well as relations between the results of the three test methods to determine the tensile strength. If different test methods are used, other correlations have to be used.

d.7.2.3. Strength development

The development of strength with time may be verified by testing concrete specimens according to clause d.7.2.1 at suitable time intervals. If the strength development of the concrete in the structure is to be verified, curing conditions simulating site exposure and deviating from those specified in clause d.7.2.1 should be employed.

d.7.2.4. Resistance to abrasion

Currently, there exists no international standard to determine the abrasion resistance of concrete. Therefore, testing should be done according to national standards or regulations.

d.7.2.5. Resistance to water penetration

The resistance of concrete to water penetration may be tested, for example, according to ISO 7031. Currently, few data are available to relate the results of experiments carried out in accordance with ISO 7031 to the long term performance of structures. If, deviating from the curing method specified in ISO 7031, the specimens are stored in water up to the time of testing, concrete may be considered to have a high resistance to water penetration if the maximum values of penetration are less than 50 mm and if the average penetration is less than 20 mm.

d.7.2.6. Density

The oven-dry density of hardened concrete is used for the classification of concrete by density. The apparent density may be determined, e.g. according to ISO 6275.

d.8. Specification of concrete

This Model Code distinguishes between two modes of concrete specification

- designed concrete mix, section d.3, definition (21)
- prescribed concrete mix, section d.3, definition (22).

d.8.1. Designed concrete mix (C I)

d.8.1.1. General requirements

The composition of designed mixes may be based on previous average production data for the materials and of the plant producing the concrete over a period exceeding at least one month and not exceeding one year. In the absence of suitable previous data, trial mixes should be made. Information on the composition of designed mixes should be made available to the user on request. The producer of the concrete is responsible for its composition, which must be such that the fresh concrete is sufficiently workable, does not segregate and can be virtually fully compacted. The resulting concrete, when hardened, must comply with the specification—usually a prescribed compressive strength. For the trial mixes, therefore, a sufficiently high strength margin should be obtained in the tests. Furthermore, the concrete must be durable and afford adequate protection to the reinforcement against corrosion. To achieve this, its composition should be in accordance with the instructions given in this appendix.

d.8.1.2. Data for specifying designed mixes

The specification of designed mixes should contain the following basic data

(*a*) strength class
(*b*) nominal maximum size of aggregate
(*c*) consistence of the fresh concrete
(*d*) limitations for the composition depending on the use of the concrete according to Table d.2 (e.g. plain concrete, reinforced or prestressed concrete and environmental conditions).

Additional data to be verified in trial tests may be specified if required for special conditions such as

(*a*) characteristics of the hardened concrete, e.g.

- (i) density
- (ii) high resistance to water penetration
- (iii) resistance to freezing and thawing
- (iv) resistance to chemical attack
- (v) resistance to abrasion
- (vi) resistance to high temperatures

(*b*) characteristics of the composition, e.g.

- (i) type of cement
- (ii) air content
- (iii) accelerated strength development
- (iv) heat development during hydration
- (v) retarded hydration
- (vi) special requirements for aggregates
- (vii) special requirements concerning resistance to alkali-silica reaction
- (viii) special requirements for the temperature of the fresh concrete.

d.8.2. Prescribed concrete mix (C II)

d.8.2.1. General requirements

In this case, the structural designer and/or the user assume responsibility for the composition of the concrete. They should check the suitability of the concrete by tests on trial mixes, unless adequate knowledge is available from previous use of similar mixes.

The trial mixes are governed by the same criteria as those which apply to the production of designed mixes (C I).

d.8.2.2. Data for specifying prescribed mixes

The specification of prescribed mixes should contain at least the following basic data

- cement content per cubic metre of compacted concrete
- cement type and strength grade
- consistence class or water/cement ratio of the fresh concrete
- type of aggregate
- nominal maximum size of aggregate
- type and, if necessary, quantity of any admixture or addition.

Additional data may be specified such as

- data with regard to concrete composition
  - additional requirements for aggregates including any special grading
  - air content of fresh concrete
  - special requirements regarding the temperature of the fresh concrete on delivery
- data with regard to the transportation and the procedures on site
  - delivery rate
  - limitation of type (agitating/non-agitating equipment), size or height of transport vehicle.

d.9. Batching and mixing of fresh concrete

d.9.1. Batching

For the batching of the basic materials the accuracies given in Table d.9 are recommended. These values apply for the total batch volume.

To achieve these accuracies for batching the measuring equipment has to be sufficiently accurate and comply with the relevant national standards or regulations.

When batching cement, the required accuracy can usually be achieved only by weighing. Alternative systems may be used, provided their reliability has been proved in practice, and the accuracy of measurement, when checked by field trials, is within the required tolerances.

Generally, aggregates should also be weighed. (For the batching of lightweight aggregates see section d.16.) If batching is carried out by volume, it is necessary to confirm frequently that the dry weight of the measured particle groups is accurate. In particular, it is necessary to carry out frequent checks and make corresponding corrections on aggregates containing variable amounts of moisture because the bulk density (unit weight) particularly of the finer particle groups will vary considerably with the water content.

Table d.9. Recommended accuracies for batching of constituent materials

Constituent	Accuracy
Cement	–
Water	–
Total aggregate	$\pm 3\%$ of required quantity
Additions	–
Admixtures	$\pm 5\%$ of required quantity

The added water can be batched by weighing or by volume. The largest permissible quantity of water depends on the required consistence. Water shall not be added to the concrete after it has left the mixer. The cement content of concrete, for which limits of the water/cement ratio are recommended (e.g. according to Table d.2), shall be sufficient to ensure that this condition is also satisfied. In this case, account shall be taken of moisture on the surface of the aggregates, the amount of water absorbed prior to placing the concrete and where appropriate, the water content of additions and admixtures if they contain water in noticeable amounts.

Admixtures in the form of powders and additions should always be measured by weight. Fluid or paste-like admixtures and additions can be measured by volume or by weight.

d.9.2. Mixing

Concrete can be mixed either in a stationary mixer or in a truck mixer. Such mixers shall be capable of achieving a uniform distribution of the constituent materials and a uniform workability of the mix within the mixing time and at the mixing capacity.

Truck mixers shall be equipped such that the concrete can be delivered in a homogeneously mixed state. In addition, they have to have suitable measuring and dispensing equipment if mixing water or admixtures are to be added on the site.

Mixing of the various constituents should continue until a uniform mixture is obtained. The duration of mixing depends on the type and composition of the concrete and also on the type and condition of the mixer. It must not be excessive so as to break up aggregates.

The temperature of the fresh concrete before casting should, if possible, not exceed 30°C and should not fall below 5°C in cold weather or frost. Frozen aggregates should be completely thawed before or during mixing. Also refer to sections d.13 and d.14.

If the temperature of the fresh concrete exceeds 30°C (e.g. when using mixing processes with steam or in hot climates) special measures are required to ensure that the concrete can still be completely compacted. Such measures include the use of a retarder or the use of a cement which is particularly suitable for these conditions. The introduction of steam at the mixer necessitates the use of special equipment and special experience.

In principle, the composition of the fresh concrete must not change after leaving the mixer.

d.9.3. Ready-mixed concrete

d.9.3.1. General requirements

When the concrete is delivered its consistence and composition shall be in accordance with the specification. It should not have segregated and must not be at an unacceptably high or low temperature (refer to section d.9.2). Furthermore, it shall remain workable for an adequate period. The period between the addition of the water and the discharge of the transport vehicle, therefore, should not be excessive.

In general, truck mixers or agitators should be completely unloaded not more than 90 minutes and other vehicles not more than 45 minutes after the addition of the mixing water. If accelerated setting of the concrete is likely to occur (e.g. in hot weather), then the period until the completion of unloading should be correspondingly shortened, unless the period in which the concrete can be satisfactorily cast is extended by the use of suitable admixtures.

If the duration of transport is very long, it may be advisable to add the water shortly before delivery. In this case, any liquid admixtures must not

be introduced prior to adding the water. However, all batching of water and admixtures at a later stage in transport should be demonstrated to be adequate by trial mixing and trial casting. The period until completion of unloading can be longer if the temperature is low or if retarders are used. Details should be taken from the appropriate national standards or regulations.

Where high range water reducing admixtures are used, the admixtures can only be added shortly before casting because of the limited duration of the fluidifying effect of such admixtures. After adding the admixture, the concrete should be re-mixed until the high range water reducing admixture is uniformly distributed and has become fully effective.

d.9.3.2. Information for user

The user of ready-mixed concrete needs information from the producer of the ready-mixed concrete on the composition of the concrete, to permit proper placing and curing of the fresh concrete as well as to assess the strength development and the durability of the concrete in the structure. This information should be given to the user either beforehand on request or at delivery.

Therefore, before discharging the concrete, the producer of the ready-mixed concrete should provide the user with a delivery ticket for each load of concrete, which contains the following basic information

- name of ready-mixed concrete plant
- serial number of delivery ticket
- date and time of loading, i.e. time of first contact between cement and water
- truck number
- name of user
- name and location of the site
- specification of the concrete as well as details or references to specifications, e.g. code number, order number
- amount of concrete in m^3
- name or mark of the certification body where appropriate.

For a designed mix the delivery ticket should, in addition, contain

- strength class
- exposure class for which the concrete is suitable
- consistence class
- type and strength class of cement
- type of aggregate
- type of admixtures and additions, if any
- special properties of the concrete.

This information may also be provided by reference to the producer's catalogue of concrete mixes which should contain information on strength and consistence class, batch weights and other relevant data.

For a prescribed mix the delivery ticket should give

- details of the composition, e.g. cement content, type of admixtures and additions, if any
- consistence class.

d.10. Handling, placing and compaction of fresh concrete

d.10.1. Handling

The method of handling (e.g. in skips, on conveyor belts or through pipes) and the composition of the concrete have to be co-ordinated in such a way that segregation is prevented.

Whereas practically any concrete can be transported in suitable skips, concrete that is to be pumped through pipes shall satisfy certain requirements as to its mix design and workability. (For pumped concrete refer to subsection d.11.5.)

Concrete moved on a conveyor belt shall also be cohesive. It is preferable to provide ploughs and devices for holding the concrete together at the point of discharge.

Concrete which is cast under water should be composed and handled in accordance with the recommendations given in subsection d.11.2.

d.10.2. Time of placing

Concrete should be placed as soon as possible after mixing or, in the case of ready-mixed concrete after delivery, so as to minimize any reduction in workability and changes in its composition.

d.10.3. Placing

Concrete should not segregate when being placed. Chutes used, when filling tall column or wall forms, should end a short distance above the place where the concrete is deposited in order to keep the concrete together. The concrete should be placed in uniform layers the thickness of which depends on the method and effectiveness of the method of compaction. Dumping the concrete in large heaps and then distributing it with vibrators is not advisable, because the concrete is liable to segregate. To avoid the formation of horizontal layers of laitance the concrete should be deposited in a sufficiently continuous manner to ensure that, while being compacted, it will be completely bonded to the previous layer of concrete.

d.10.4. Compaction

The concrete should be compacted as completely as possible, so that it contains a minimum amount of entrapped air. Depending upon the workability of the concrete and the type of structural component, the concrete may be compacted by vibration, rodding, stamping, hammering of the formwork etc. Concrete of stiff, plastic or semi-fluid consistence should generally be compacted by vibration. The compaction of fluid concrete should be carried out by rodding or, if segregation is not probable, by light vibration.

When vibrators are used vibration should be applied continuously during the placing of each batch of concrete until the expulsion of air practically ceases and in a manner which does not promote segregation.

Particular attention should be paid to ensure that the reinforcing bars are embedded in dense concrete. While placing and compacting the fresh concrete care should be taken to avoid displacement and damage of reinforcement, tendons, ducts, anchorages and formwork.

Concrete which has already been compacted can be improved by revibration at a later time, as long as it is still workable. Revibration tends to close plastic shrinkage and settlement cracks and cavities under horizontal reinforcing bars.

Where a special surface finish is required, this should be specified additionally.

d.10.5. Construction joints

Construction joints are formed in places where the concreting work has to be interrupted for practical reasons. The number of joints should be kept to a minimum since they can have low tensile and shear strengths thus reducing the load-bearing capacity near the joint. Furthermore, there is a

risk that faulty workmanship may impair the watertightness of the concrete near the construction joint; this can also reduce the protection of the reinforcement against corrosion.

If possible, construction joints should be located at places where they will not be heavily stressed or where a joint is required for other reasons. Horizontal joints should not be placed in positions intermittently below the water level.

To provide a good bond between the old and the new concrete any laitance on the hardened concrete should be removed before the fresh concrete is cast.

The older concrete, if dry, should be wetted before further concrete is placed. At the time of concreting, however, the older concrete should be somewhat dry on the surface and slightly absorptive, whereas the core should still be wet, i.e. the concrete should be saturated but surface dry.

d.11. Concrete for special manufacturing or casting conditions

d.11.1. Concrete containing a combination of admixtures

For some applications (e.g. for mass concrete, for concrete with a high resistance to freezing and thawing and de-icing chemicals or for concreting at a very slow rate) frequently the simultaneous use of two or more different admixtures is advantageous. Different admixtures may influence each other's effectiveness with regard to the properties of the fresh or hardened concrete, and in some instances may not be compatible, e.g. some high range water reducing admixtures and air entraining admixtures.

Also setting time and strength development may be significantly altered by combinations of admixtures. Therefore, in such cases special knowledge is required, and trial tests with the intended combination of admixtures are mandatory. Furthermore, the manufacturers of the admixtures should be consulted if any doubts exist concerning the compatibility of the admixtures.

d.11.2. Concrete cast under water

Concrete placed under water to form load bearing or impermeable structures should flow as a coherent mass, so that it will attain a dense structure even without compaction and does not segregate.

The following recommendations apply to the composition of concrete cast under water.

(*a*) *Consistence.* A semi-fluid consistence corresponding to the slump classes S3 or S4 in Table d.5 should be used. A slump of about 150 mm or greater is advisable. The optimum consistence also depends on the manner in which the concrete is cast. Concrete intended for pumping, normally should be somewhat stiffer to prevent a blockage of the pipes. Fluid mixes, however, can be pumped satisfactorily if suitable admixtures (high range water reducing admixtures, water retaining admixtures) are added.

(*b*) *Aggregates.* To achieve good workability combined with a low water/cement ratio and close, compacted texture without additional means of compaction, the use of aggregates of rounded or cubic particle shapes and a smooth surface is preferable. In general, continuous gradings are preferred, because the danger of segregation is less than with gap graded mixes. Grading curves near the centre of zone 3 in Figs d.2 to d.5 are suitable while the proportion of fines up to 4 mm in size should lie near line B. Maximum coarse aggregate sizes exceeding approx. 32 mm can lead to difficulties in casting. In some cases a maximum aggregate particle size of less than 32 mm is advisable.

(*c*) *Cement*. Recommendations on the required cement content have not yet been agreed on. High cement contents of about 350–400 kg/m^3 are often recommended in the literature to improve the cohesion of the fresh concrete and thus diminish the danger of loss of cement by leaching, as well as to ensure that an adequate amount of cement remains in the hardened concrete, in spite of the leaching which will inevitably occur. It also has been proposed that the cement content should be limited to 325 kg/m^3 because high cement contents can encourage the formation of wide thermal cracks. If the aggregate does not contain a sufficient quantity of ultrafines, it may be necessary to introduce additional ultrafine material to optimize the cohesion of the fresh concrete. The use of cements other than Portland cements (types CE II–IV) may be appropriate for concrete cast under water, in order to increase its resistance against chemical attack and to reduce heat of hydration.

Unless special admixtures or additions are used which improve concrete cohesion in the fresh state, concrete cast under water should not fall freely through the water. Otherwise it may be leached out and become segregated. The following processes have been particularly effective when casting concrete under water.

(*a*) *Tremie (pipe) method.* The concrete is placed through vertical pipes, the lower end of which is always inserted sufficiently deep into the concrete which has been placed previously but has not set. The concrete emerging from the pipe pushes the material that has already been placed to the side and upwards and thus does not come into direct contact with the water.
(*b*) *Direct placement with pumps.* This is a further development of the tremie method. As in the case of the tremie method the vertical end piece of the pipe line has always to be inserted sufficiently deep into the previously cast concrete and should not move to the side during pumping.
(*c*) *Special mix design.* This includes the use of admixtures or additions which assure sufficient cohesion of the fresh concrete. Under these conditions, the concrete may be freely dropped through the water.

Such methods and concrete compositions may also be used for casting concrete in bentonite.

d.11.3. Shotcrete

Shotcrete, or gunite, is the process by which concrete or mortar is sprayed onto a surface to produce compacted material which is self-supporting. There are two basic systems. In the dry process the pre-mixed dry materials are mixed with the water at the spray gun whereas in the wet process the mixing takes place earlier.

The advantage of shotcrete is that it requires only an inner form (or an existing surface). Therefore it is particularly suitable for the construction of curved sections, for tunnel linings or coatings and for the repair or strengthening of structural elements.

When defining the composition of shotcrete, distinction has to be made between

- the initial mix which corresponds to the original shotcrete mixture before it is transported to the shotcreting equipment
- the jetted mix which is the shotcrete which leaves the nozzle
- shotcrete, which is the material in place; it differs from the jetted mix by the rebound which occurs during jetting.

Careful consideration of the mix design is necessary by the shotcrete contractor to ensure good adhesion, proper compaction and satisfactory properties of the hardened material. The water/cement ratio of the in-place concrete or mortar is usually in the range 0.35–0.5. Maximum aggregate size for sprayed concrete is about 20 mm. The density of the in-place concrete is similar to equivalent, well compacted, conventional concrete.

Because of the high particle velocities involved in this technique, special consideration has to be given to the safety of personnel.

The various steps in producing and placing shotcrete and the substantial differences between the composition of shotcrete and the initial mix require careful control and testing during the various steps of the entire operation.

d.11.4. Vacuum dewatering

Vacuum dewatering is a technique for removing water from the surface of freshly cast concrete. It results in much faster stiffening of the fresh concrete and considerable improvement in the physical properties of the hardened concrete.

Vacuum dewatering is most commonly used in conjunction with power trowelling for the production of smooth, hardwearing floor surfaces. In this application, vacuum dewatering enables the usual delay between casting and surface finishing to be reduced from several hours to less than one hour, which can be a considerable advantage in cold weather.

Vacuum dewatering is particularly useful with concrete of fluid consistence but is not suitable for use with air-entrained concrete.

d.11.5. Pumped concrete

Materials for pumpable concrete should be batched consistently and uniformly, and should be thoroughly mixed. Quality control of the ingredients and of the batching and mixing is essential for smooth-running pumping operations and for good quality of the pumped concrete.

The maximum particle size of angular aggregates should be limited to one-third of the internal diameter of the pipe or hose, and, in the case of well rounded aggregate, to 40% of the pipe diameter.

Adequate and uniform grading of the aggregate during the whole pumping operation is very important. Grading should preferably be continuous and should follow the grading curve B in Figs. d.2 to d.5 as closely as possible. Gap-graded aggregate should be used with caution, since it is prone to segregation.

The content of ultrafine materials should be high enough to provide good cohesion, but the excess may have deleterious effects on the fresh or hardened concrete (see clause d.6.5.1.4). Attention should also be paid to the portion of the fine aggregate passing sieve size 0.25 mm, which should be between 15 and 30%, and to that passing sieve size 0.125 mm, which should not be less than 5 to 10% of the total volume of the sand. If the fine aggregate is deficient in either of these two sizes the above recommendation can be met by blending with selected finer sands or with an adequate type of addition.

Lightweight aggregates should be pre-wetted in order to offset excessive absorption under the pressure exerted by pumping and the stiffening of the concrete during pumping operation.

The required cement content depends on the same basic principles valid for conventionally placed concrete, though slightly higher amounts of cement may be necessary due to the higher consistence classes and the higher amount of ultrafines used for pumping.

Admixtures which increase workability usually improve pumpability by providing additional lubrication, reduce segregation and decrease bleeding.

Air-entraining admixtures also improve workability, but too large a quantity of entrained air may make pumping difficult. Set-retarding admixtures may be used when pumping long distances and in hot weather.

The consistence of fresh concrete for pumping, measured by the slump test, should be in the plastic range i.e. in classes S2 and S3. Too fluid a consistence may produce excessive bleeding, and, therefore, cause blocking in the pump line. Maintaining control of proper and uniform consistence is essential for a smoothly running pumping operation.

The suitability of the concrete mixture should be verified by trial mixes and by performing a pumping test.

Pipes for pumping should not be made of materials having a deleterious effect on concrete; aluminium alloy, for example, may cause the generation of hydrogen bubbles and thus a reduction of concrete strength.

Before starting the pumping operation the pipe line should be lubricated with a properly designed mortar, which should be wasted and not used in the concrete placement, or with a batch of the regular concrete with the coarse fraction omitted.

d.12. Curing and protection

d.12.1. General considerations

In order to obtain the potential properties to be expected from the concrete, thorough curing and protection over an adequate period of time are vital. It is essential that curing and protection start immediately after compaction of the fresh concrete.

In this context curing is a measure to avoid premature drying of the concrete and to provide the cement paste in the concrete with a sufficient amount of water over a sufficiently long period of time to achieve a high degree of hydration within its mass and particularly in its surface layers. In addition, curing includes measures against the effects of sunshine or wind and the prevention of cracking due to early shrinkage.

Insufficient curing often has only a minor effect on the strength development of the concrete in the structure with the exception of thin sections because the core of a thicker concrete section maintains a sufficiently high moisture content for a prolonged period of time even without curing. However, lack of curing is detrimental to the durability of a concrete structure which is primarily controlled by the properties of the surface layers. Premature drying results in a permeable surface layer with little resistance to the ingress of aggressive media.

Protection is a measure against other external effects which may harm the young concrete such as leaching due to rain or flowing water, rapid cooling or freezing, thermal stresses due to heat of hydration, vibration or impact. Protection against freezing is dealt with in section d.13.

d.12.2. Methods of curing

The principal methods for the curing of the concrete are

- keeping the formwork in place
- covering with plastic films
- placing of wet coverings
- sprinkling with water (at temperatures above freezing)
- application of curing compounds which form protective membranes.

These methods may be used separately or in combination.

Not all methods of curing are equally effective. Therefore, it is essential that the effectiveness of the method chosen is controlled e.g. by frequent inspection. In general, those methods where water is added result in a

denser pore structure of the concrete than methods which only prevent the concrete from drying. Nevertheless, sprinkling of cold water on a concrete surface which is warm due to the heat of hydration may cause severe temperature stresses and early cracking of the concrete surface layers.

d.12.3. Duration of curing

d.12.3.1. Parameters influencing the duration of curing

The concrete has to be cured until its surface layers are sufficiently impermeable. This generally means that it will also attain the required strength. Therefore, the duration of curing depends on the following.

- *Curing sensitivity of the concrete as influenced by its composition.* The most important characteristics of concrete composition with respect to curing are the water/cement ratio, the type and strength class of cement, as well as type and amount of additions. Concrete with a low water/cement ratio and made of a rapidly hardening cement e.g. R- or RS-cements according to clause d.4.2.1 reach a required level of impermeability more rapidly and, therefore, need less curing than concretes with a higher water/cement ratio and made of cements which hydrate slower such as SL-cement according to clause d.4.2.1 or concretes containing higher amounts of Type II-additions.

  Also the resistance of the concrete to a particular exposure condition as influenced by the type of cement or additions should be taken into account. A lower resistance to e.g. carbonation which is characteristic for some of the blended cements CE II–CE IV according to clause d.4.2.1 can be offset by the choice of a lower water/cement ratio or prolonged curing.
- *Concrete temperature.* Due to the heat of hydration generated by the reaction between cement and water the concrete temperature may increase, thus accelerating hydration. Therefore, the higher the temperature particularly of the surface layers of the concrete, the shorter the required duration of curing. The temperature of the concrete depends on the temperature of the ambient air, strength grade and amount of cement, the dimensions of the structural member and the insulation properties provided e.g. by the formwork. Thus thin concrete sections without thermal insulation exposed to low ambient temperatures during curing and made of cements with a low heat of hydration need particularly careful curing.
- *Ambient conditions during and after curing.* A low relative humidity of the ambient air, sunshine and high winds accelerate drying of the unprotected concrete at an early stage of hydration. Therefore, under such conditions prolonged curing is required, because after termination of curing the surface layers of the concrete dry out rapidly, and hydration will no longer continue. On the other hand when concreting in a humid environment at moderate temperatures curing will at least partially be provided by the surrounding atmosphere.
- *Exposure conditions of the finished structure in service.* The more severe the exposure conditions as given in Table d.1, the longer the required duration of curing.

Thus, an estimate of the required duration of curing is a complex problem. The best approach is to define limiting values of permeability of the concrete surface layers which have to be reached before curing can be terminated. These value should depend on the exposure condition of the structure in service as well as on the type of cement but not on water/cement ratio, strength class of the cement and concrete temperature. At this time neither methods to measure surface permeability nor limiting values of

permeability are generally accepted. Therefore, the required duration of curing has to be estimated on the basis of the parameters given above.

Some of these parameters are interrelated particularly with regard to concrete temperature so that a reliable estimate of the required duration of curing necessitates preliminary experiments on the concrete in question and continuous measurements of concrete temperature on site. Such an approach which is based also on theoretical considerations is given in 'Durable Concrete Structures—CEB Design Guide'. Rules of thumb for a quick estimate are given in the following section.

d.12.3.2. Estimates of the duration of curing

In Table d.10 minimum durations of curing are proposed for concrete members subjected to exposure condition 2a; 2b; 4a and 5a according to Table d.1. In Table d.10 distinction is made according to expected ambient conditions during curing and during the period immediately after curing and according to the rate at which the concrete reaches a certain impermeability. This rate depends on the water/cement ratio and strength class of cement as proposed in Table d.11. There, reference is made to the general cement classification given in clause d.4.2.1.

Table d.10. Minimum duration of curing in days for $T > 10°C$ for exposure classes 2a; 2b; 4a and 5a, according to Table d.1

Expected ambient conditions during and after curing	*Rate of development of impermeability of concrete*			
	Very rapid	*Rapid*	*Medium*	*Slow*
I No direct sunshine, relative humidity of surrounding air RH > 80%	*1*	*2*	*3*	*4*
II Exposed to medium sunshine or medium wind velocity or relative humidity RH: 50% < RH < 80%	*2*	*3*	*4*	*5*
III Exposed to strong sunshine or high wind velocity or relative humidity RH < 50%	*3*	*4*	*6*	*8*

Table d.11. Rate of development of impermeability of concrete

Rate of development of impermeability	*w/c*	Class of cement*
Very rapid	0.5–0.6	RS
	< 0.5	RS; R
Rapid	0.5–0.6	R
	< 0.5	N
Medium	0.5–0.6	N
	< 0.5	SL
Slow	All other cases	

* Refer to clause d.4.2.1.

Table d.10 is valid for concretes made of Portland cements (CE I) and for a reference temperature of the concrete of 20°C. For temperatures of the ambient air during curing < 10°C the duration of curing should be increased. Then the required increase of curing time may be determined on the basis of the maturity concept given in clause 2.1.8.2 of this Model Code. As a rough guide for a concrete temperature of 10°C the required duration of curing is about twice the required curing time for a concrete temperature of 20°C. For a concrete temperature of 30°C it is only about half the curing time at 20°C. Therefore, thermal insulation of the concrete during curing may be an effective method to reduce curing times, particularly for concrete made of slowly hydrating cements. In such cases, however, attention has to be paid to thermal stresses which may be developed when the thermal insulation is removed. Refer to subsection d.12.4.

Concretes made of cements containing high amounts of constituents other than Portland cement clinker (CE II 32.5; CE III 32.5; CE IV 32.5) and concretes containing Type II additions in amounts which approach the maximum values given in clause d.6.3.3.2 are more sensitive to curing than concretes made of Portland cements with the same water/cement ratio. Therefore, for such concretes the duration of curing should be increased by 1 to 2 days beyond the values given in Table d.10 if during curing the concrete is exposed to conditions II and III. For condition I the rate of drying after curing is so low, that even the surface layer of the concrete will continue to hydrate for some time after the termination of curing.

Depending on the type and use of the structural element (e.g. the intended finish) the minimum duration of curing given in Table d.10 should also be applied for exposure class 1. Where the concrete is exposed to severe abrasion or to severe environmental conditions (exposure classes 3, 4b, 5b and 5c according to Table d.5) the length of curing given in Table d.10 should be increased by 3 to 5 days depending on the ambient conditions during curing according to Table d.10.

d.12.4. Protection against thermal cracking of surface

The hardened concrete has to be protected against damaging effects caused by heat of hydration generated in the concrete.

Where no cracking is permitted, adequate measures should be taken to ensure that the tensile stresses caused by temperature differences are less than the tensile strength at the time these stresses occur. This generally requires a careful numerical analysis.

To avoid cracking caused by a temperature rise of the concrete at its surface, the temperature difference between the centre and the surface should be less than 20°C. Details on measures to prevent cracking by protection of the young concrete are given in 'Durable Concrete Structures —CEB Design Guide'.

d.12.5. Heat treatment

The rate at which concrete hardens can be accelerated by heat treatment, because a rise of temperature during hardening, within certain limits, increases the early strength; the final strength, however, may be somewhat less than that of concrete stored at normal temperatures. Decisive factors in this respect are the age of concrete at the commencement of heat treatment, the rate of temperature rise, the level of temperature, the duration of heat treatment and the rate of cooling. The successful application of heat treatment depends on the type of cement used, but generally applicable rules cannot be given. Hence, before commencing production, the materials to be used and the method of treatment employed should be checked for their suitability through trial mixes.

Unless there is sufficiently documented positive experience with the con-

ditions of a particular heat treatment of a concrete of given composition and constituent materials, particularly the cement, it is recommended that for curing of concrete members which will be subjected to exposure classes 2 to 5 (Table d.1), the following limitations with regard to heat treatment (steam curing) are observed.

- Concrete temperature during the first 3 hours after mixing should not exceed 30°C and should not be higher than 40°C during the first 4 hours.
- The rate of temperature increase should not exceed 20 K/h.
- The average maximum temperature of the concrete should not exceed 60°C (individual values < 65°C).
- The concrete should be cooled at a rate not exceeding 10 K/h.
- The concrete should be protected against moisture loss throughout the curing procedure and while cooling.

These limitations in the temperature regime are given to prevent chemical reactions some time after the heat treatment, generally referred to as secondary ettringite formation. They are associated with a volume increase and in turn cracking and disruption of the concrete. Since these reactions depend on the composition of a particular cement a more severe heat treatment than given above does not necessarily lead to such damaging reactions.

Precautions required when introducing steam into the mixer are discussed in section d.9.2.

Heat treatment can influence the final properties of the hardened concrete (e.g. ratio of tensile strength to compressive strength, deformation properties, durability). This should be taken into account in design.

d.13. Concreting in cold weather or frost

The hydration of the cement paste is retarded at lower temperatures. Apart from this, frost can permanently impair immature concrete if the water contained in the pores freezes and disrupts the concrete. Therefore, at low ambient temperatures appropriate measures should be taken to ensure that the rate of hardening is adequate and that frost damage is prevented.

Whenever there is a risk of frost or prolonged freezing, the fresh or very young concrete should be protected from freezing by arrangements for covering and/or insulating the newly placed concrete or by enclosures for heating the air surrounding the newly cast structural element. Especially in the latter case, adequate provisions for maintaining proper moisture conditions are essential.

Before being exposed to freezing temperatures, concrete should have a compressive strength of at least 5 MPa. The time required to reach this minimum freezing strength will be the shorter the higher the temperature of the concrete and the lower the water/cement ratio. For a more accurate estimate of this time refer to 'Durable Concrete Structures—CEB Design Guide'.

The drop in temperature of any portion of the concrete should be gradual, not exceeding 10°C in massive structures and 20°C in non-massive structures throughout the first 42 hours after the end of protection. The safe removal of formwork and its supports is controlled by the proportion of design strength which should be attained at the time of stripping forms or removing their supports.

The minimum temperature of fresh concrete at the time of placing should not be lower than + 5°C. However, when concrete is cast in thin sections this minimum temperature should preferably be at least 10°C. The initial temperature of fresh concrete as mixed should be raised to offset heat loss

between concrete mixing and placing, but it should in no case be higher than 30°C.

The desired temperature of the fresh concrete can be achieved by heating the constitutive materials. Mixing water may be heated up to 100°C, and the average temperature of aggregates for an individual batch should not exceed 65°C. To avoid the possibility of flash set, water and aggregate should be first mixed together in such a way that the temperature of the mixture is reduced to less than 40°C, and only after that may cement be admitted into the mixer. The aggregate should be free of lumps of ice and snow.

The subgrade on which fresh concrete is placed should not be frozen, and its minimum temperature should be at least 3°C.

For details reference is made to the appropriate provisions of the various national standards.

d.14. Concreting at high temperatures

The properties of concrete can be influenced unfavourably by hot weather. In concrete technology hot weather is defined as any combination of high temperature, low relative humidity and high wind velocity tending to impair the quality of fresh and hardened concrete or otherwise resulting in abnormal properties. High temperatures of fresh concrete accelerate setting, increase the rate of hydration and the water demand, and lead to diminished final strength. Hot weather also causes difficulties during placing and the formation of plastic shrinkage cracks in young concrete.

Therefore, every effort should be made to ensure that the temperature of the concrete at the time of placing is lower than 30°C to 35°C for normal structures, and less than 15°C for mass concrete. In cases where severe internal stresses or restraint may develop due to differences between the increased temperature caused by heat of hydration and the temperature of the surrounding air, the temperature of the fresh concrete at the time of casting should be controlled so as to limit this temperature difference. Also refer to subsection d.12.4.

This should be achieved by

- cooling of the mixing water
- reducing the aggregate temperature by shading the stockpiles and/or sprinkling or spraying coarse aggregate fractions
- keeping the cement temperature low; if the initial temperature of the cement is sufficiently low, this may be achieved by insulating the walls of the silos
- shading and/or sprinkling the batching, mixing, delivery and placing equipment
- good co-ordination and speeding-up of placement and finishing operations
- proper planning of the time of casting, so that the concrete is placed when the ambient temperature is as low as possible.

Curing should be carried out in accordance with section d.12. When concreting at high temperatures moist curing is particularly suitable. It should continue without interruption for a minimum of 7 days. For flat structures (slabs, pavements), however, the use of an appropriate membrane-forming curing compound which is evenly applied immediately after completion of the finishing operations often is more practical.

During placing and finishing operations, particularly flat structures should be protected from wind and sunshine by windbreaks and/or shades,

and any excessive evaporation of water causing early shrinkage cracking should be prevented by cooling and moistening or fogging the surrounding air.

d.15. Retempering

Retempering is the addition of water or cement paste with an appropriate water/cement ratio, or of a high range water reducing superplasticizing admixture, and the remixing of concrete which, on arrival at the construction site, has lost so much workability that it has become unplaceable or unusable.

Retempering with water shall be prohibited outright since it is very likely to increase the original water/cement ratio, thus reducing the quality of the hardened concrete. It may be tolerable only if the effective water/cement ratio remains within specified limits after taking into account estimated amounts of water already chemically combined or evaporated.

Retempering with a high range water reducing superplasticizing admixture is much more convenient, because it may even allow a reduction of the water/cement ratio of the initial mix. In such cases a proper amount of admixture should be added directly into the truck mixer where mixing should continue until a uniform consistence of the batch is achieved. Repeated retempering is feasible, but its effectiveness diminishes. Therefore, it is not advisable to retemper more than twice. Attention should be paid to the fact that the use of high range water reducing superplasticizing admixtures may cause a loss of entrained air.

d.16. Structural lightweight aggregate concrete: special factors

d.16.1. Requirements for aggregates

Lightweight aggregates should satisfy the requirements contained in the relevant standards.

Water absorbed by the aggregates can affect the resistance of the concrete to damage due to freezing and thawing, its fire resistance and thermal resistivity. Where these properties are of importance, suitable measures should be taken to prevent the absorption of excessive quantities of water by the aggregate as it may take a prolonged period, several years in some cases, before the absorbed water can be liberated.

d.16.2. Mix design

The composition of lightweight aggregate concrete should be based invariably on trial mixes, unless the design is derived from previous experience with the same constituent materials (C I according to subsection d.8.1 or C II according to subsection d.8.2). For acceptance of a mix design the results of the tests on the trial mixes should indicate that a strength appropriate for the prescribed strength class, the required density and other required properties can be attained with adequate certainty and that the workability of the concrete will be satisfactory.

d.16.2.1. Grading of aggregates

Lightweight aggregates shall be delivered and stored separately in a sufficient closed graded range of sizes to allow for the fact that lightweight aggregate particles may vary in some of their properties with the size of the aggregate particles (e.g. particle density, strength, water absorption). Accidental mixing of different gradings or segregation within a group

may affect the workability, strength and density of lightweight aggregate concrete. It is recommended that the maximum particle diameter should not exceed 25 mm, in order to avoid segregation and as a consequence reduction in the strength of the concrete.

Advice on the grading of the aggregates is given in clauses d.6.5.1.2 and d.6.5.1.3 for continuous and gap-graded mixes.

d.16.2.2. Cement content

The required cement content can be determined from trial mixes. For reinforced and prestressed lightweight aggregate concrete the cement content used should not fall below certain minimum values (e.g. 300 kg/m^3). Nor should the cement content be allowed to be too high (more than 500 kg/m^3) since—as for all other types of concrete—this can lead to microcracking due to excessive heat development, especially in large components and possibly to excessive shrinkage and creep, in the absence of adequate curing and other preventative measures.

d.16.3. Consistence

In general it is recommended that, immediately before the concrete is cast, its consistence should lie in the plastic range. Lightweight aggregate concrete having a stiff consistence should only be used in special circumstances since it tends to be more difficult to place and compact.

On the other hand it should be borne in mind that semi-fluid and fluid lightweight aggregate concrete tends to segregate owing to rising of the lighter coarse aggregate particles.

d.16.4. Prewetting and batching of aggregates

Water absorption by the aggregates may cause a reduction of the effective water content of the cement paste. The effective water content of the cement paste is important with respect to the evolution in time of workability, to strength and to durability requirements. In order to fix the total amount of mixing water (w_{tot}) and the effective water content (w_{ef}) or the water/cement ratio (w/c) one of the following procedures is proposed.

Dry aggregates

In this case two methods can be applied.

- The dry aggregates and the sand are mixed during 1 min with 40–60% of the total amount of water. This total amount is determined by adding conventionally to the effective water in the cement paste, the amount of water absorbed by the aggregates in 30 minutes (measured on a separate sample).

$$w/c = (w_{tot} - w_{30\,\mathrm{min}})/c$$

- The dry aggregates remain in water during 30 minutes. Before mixing the aggregates are drained for 5 minutes. Only the supplementary water is added during mixing.

$$w/c = w_{add}/c$$

Wet aggregates

After the determination of the moisture content (m), comparison with the absorption after 30 minutes indicates whether supplementary water has to be added

$$w/c = [w_{tot} - (w_{30\,\mathrm{min}} - m)]/c \quad \text{if } m < w_{30\,\mathrm{min}}$$

$$w/c = w_{add}/c \quad \text{if } m > w_{30\,\mathrm{min}}$$

Trial mixes with a particular aggregate are recommended to fix the total amount of mixing water leading to the desired workability and strength.

If the moisture content of the aggregate is highly variable, batching the coarse aggregate by volume may be preferable. The moisture held in the aggregate particles need not be taken into account but of course, the mixing water (i.e. the quantity of water to be added at the mixer) must be considered. However, when batching by weight, the water held in the aggregate should be taken into account.

d.16.5. Mixing

The mixing process should continue long enough to ensure that the resulting fresh concrete is homogeneous. The period required may be longer than that needed for normal weight concrete. However, in judging the duration of the mixing process, the enhanced friability of some types of lightweight aggregate (e.g. most foamed slags) should not be overlooked.

Admixture should not be added to the mix before the aggregate particles are sufficiently wetted, to prevent a part of the admixture being absorbed by the aggregate particles, thus reducing the effect of the admixture.

d.16.6. Ready-mixed concrete

In practice lightweight aggregate concrete should be transported in truck mixers so that it can be intensively re-mixed immediately before delivery on the site.

Owing to the higher water absorption of lightweight aggregates, loss of workability during transport and handling can be higher for some types of lightweight aggregate concrete than for concrete made with normal weight aggregate.

It is recommended that trial mixes should be produced if a particular lightweight aggregate is to be used for the first time for a ready-mixed concrete. For these trial mixes account should be taken of the proposed working procedure, including length of haul, handling conditions and casting procedures.

d.16.7. Handling

When handling lightweight aggregate concrete, the greater tendency of some types of lightweight aggregate to segregate compared with normal weight aggregate should be taken into account. This tendency will be enhanced where the concrete is of fluid consistence and the density of the aggregate particles is low (less than 1000 kg/m^3). The cohesion of fresh concrete can be improved, where necessary, by using admixtures, such as water reducers, air-entraining agents and stabilizers or by the use of certain additions.

When pumping lightweight aggregate concrete, careful attention should be given to the mix design and the type of pumping equipment and aggregate to be used (e.g. natural sand should preferably be used when pumping is considered), in order to reach the required distance and lift. Depending on the type of pumping equipment and aggregate used, the effective pumping distance and lift can be less than those achieved in the case of concrete with normal weight aggregate.

d.16.8. Casting

For some types of very lightweight aggregate concretes compaction may be more difficult to achieve, than for concrete made with normal weight aggregates. Consequently the positions where concrete is placed in the formwork and the points where vibration is effected should be more closely

spaced. Lightweight aggregate concrete should always be compacted by means of vibration, as indeed is the case for normal weight concrete.

In most cases the effective radius of action for poker vibrators in lightweight aggregate concrete is only about half that observed for concrete made with normal weight aggregates. Consequently the customary distances between immersion points should be approximately halved. For some types of lightweight aggregate concrete, it may be more difficult to obtain a satisfactory surface finish because the coarse lightweight aggregates particles tend to float. This can be alleviated by using surface vibrators or by treating the surface with a roller fitted with a perforated drum, thus pushing the larger aggregate particles down while allowing the fines to rise.

d.16.9. Curing

Because of the large quantity of water absorbed by lightweight aggregates, concrete made with such aggregates responds well to curing. As evaporation takes place on the surface of the concrete, water absorbed by the aggregate is released and transferred to the matrix. Thus, water is automatically available for hydration by continuous replacement for a period which will depend on the ambient conditions.

In temperate climates it is sufficient to ensure proper hydration without the use of external devices to prevent evaporation, such as damp sacks or a plastic membrane, usually required in the case of concrete made with normal weight aggregates. In hot climates, however, careful attention should be given to the protection of the concrete surface against rapid drying out.

Curing of lightweight aggregate concrete may need to be more closely controlled than that of concrete made with normal weight aggregates, particularly if the lightweight aggregate is very dry and continues to absorb much water even after finishing. With regard to the duration of curing the recommendations given in section d.12 are applicable.

The temperature increase due to heat of hydration tends to be higher for lightweight aggregate concrete, but owing to the lower thermal expansion coefficient of lightweight aggregates and closer compatibility of the moduli of elasticity of aggregate and matrix, the extent of microcracking is generally less in lightweight aggregate concrete.

d.17. Production of high strength concrete

d.17.1. Principles

The scope of the previous model code MC 78 was limited to strength grades up to C 50. In the following sections some specific facts relating to the production of cement based concrete of higher strength, frequently referred to as high strength concrete, are dealt with. Such concretes may be advantageous not only because of their higher strength, but also because of their lower permeability thus enhancing the durability of concrete structures.

A strength of concrete in excess of 50 MPa is reached by a reduction of the water/cement ratio or of the water/cement + effective pozzolan ratio to values below 0.40. This is accomplished by the use of high range water reducing superplasticizing admixtures. Sometimes highly reactive additions such as silica fume or some pulverized fuel ashes substantially contribute to the development of high strength.

In general no particular equipment, batching and mixing methods or unusual admixtures or additions are necessary to produce such concretes.

d.17.2. Choice of materials

d.17.2.1. Cements

The cements used for high strength concrete must satisfy the general requirements for ordinary concretes.

Since in many instances high amounts of high strength Portland cements are used, particular attention should be paid to the development of heat of hydration and its consequences.

d.17.2.2. Aggregates

The aggregates for high strength concrete should satisfy the requirements laid down for aggregates for ordinary concrete. When choosing suitable aggregates, particular attention should be paid to possible alkali-silica reactions because of the high cement contents of high strength concretes.

d.17.2.3. Admixtures

In most instances, water reducing or high range water reducing superplasticizing admixtures are used to reduce the water demand for a given consistence of the fresh concretes.

d.17.2.4. Additions

In many instances, condensed silica fume or highly reactive pulverized fuel ashes with a sufficiently high specific surface area are used as additions.

d.17.2.5. Mixing water

The requirements are identical to those for ordinary concrete.

d.17.3. Consistence of fresh concrete

Whereas pulverized fuel ash in many instances reduces the water demand, the use of silica fume results in a substantial increase of the water demand. For this reason high range water reducing superplasticizing admixtures are often used in concretes containing silica fume.

For the characterization of consistence the Vebe test proved to be particularly suitable. In many practical applications it became apparent that high strength concrete can also have a fluid consistence in the fresh state. For such concrete the flow table is particularly suitable to measure concrete consistence.

d.17.4. Concrete composition

In principle, there are no differences between the principles for the composition of ordinary and of high strength concretes. However, particular attention should be paid to the proper grading of aggregates.

d.17.5. Batching and mixing

For batching and mixing of high strength concrete the rules laid down for ordinary concrete apply (section d.9).

d.17.6. Handling, placing and compaction

Silica fume reduces the tendency of concrete to segregate. In many instances high strength concrete is very suitable for pumping. Otherwise, the same rules for handling and placing apply as for ordinary concrete (section d.10).

In order to achieve the required high strength, proper vibration to remove entrapped air is of particular significance.

d.17.7. Curing

Because of the low water/cement ratio early loss of water may stop hydration of high strength concrete, and the low bleeding rates may cause plastic shrinkage. Therefore, the concrete has to be cured carefully, employing curing methods where water is added rather than using methods which only prevent moisture loss. Irrespective of subsection d.12.3 a minimum curing period of 3 days is recommended in all cases.

d.18. Personnel, equipment and installations

d.18.1. General requirements

Batching and mixing, handling and placing, curing as well as testing and the quality control of concrete require reliable supervising personnel with a successful service record in concrete and reinforced concrete construction, as well as sufficient knowledge and experience, so that the successful execution of construction work is ensured. Making and placing the concrete should be carried out by trained and qualified personnel (concretors).

d.18.2. Supervisor on the construction site

During the course of construction, the contractor or the supervisor assigned by the contractor or his/her knowledgeable representative have to be present on the site during construction. The supervisor has to make sure that the work is executed according to the applicable specifications, drawings and regulations. In particular he/she is responsible for

- the dimensions of the structural members as planned
- the safe construction and stiffening of the formwork
- quality of the constituent materials as specified
- conformity of the reinforcement with the drawings
- the proper choice of the time of stripping the forms
- prevention of overloading finished structural members
- elimination of damaged prefabricated members which could reduce the load carrying capacity of the structure
- correct installation of auxiliary supports where needed.

For the batching and mixing, handling and placing, compaction as well as curing, all installations and equipment, which are required for a high quality concrete with uniform strength as pointed out in section d.9 through d.17, have to be available and properly maintained.

d.18.3. Supervisor for precast concrete and ready-mixed concrete plants

During the working hours the supervisor or his/her knowledgeable representative have to be present on the plant. He/she has to fulfil similar tasks as have been pointed out in subsection d.18.2 as far as they apply to the work in a plant.

In addition, the supervisor is responsible for

- additional requirements regarding equipment and installations in such plants
- observation that only such ready-mixed concrete or precast concrete units leave the plant which satisfy all technical requirements
- delivery tickets which have to contain all necessary information.

d.18.4. Permanent concrete laboratory

Contractors who use concrete with special properties according to subsection d.6.6 or according to section d.11, as well as ready-mixed concrete and precast concrete plants should have access to a permanent concrete laboratory, which is equipped with all testing apparatus and installations necessary for preliminary experiments, as well as for quality control. The laboratory has to be situated such that a close co-operation with the construction site is possible. If the contractor makes use of an external testing laboratory, a contract specifying all testing and quality control measures to be conducted by the laboratory has to be issued. Such contracts should be valid for a prolonged period of time.

Production control of a plant or of a site should not be carried out by a concrete laboratory which at the same time carries out production control measures for the constituent materials.

The concrete laboratory has the following tasks

- trial testing of the concrete
- quality control of the concrete, including strength development, as far as these tests are not conducted by the personnel on the site
- control of equipment on the site and in the plant prior to commencement of the concrete work
- continuous control as well as advice during manufacturing, handling and curing the concrete; records have to be kept about the results achieved
- evaluation and judgement of the results of all tests carried out for the individual plants and sites, and reports of the results to the contractor and his supervisor or the supervisor of a plant
- training of personnel on the site and of the plant.

The concrete laboratory has to be headed by an expert knowledgeable in concrete technology and concrete production. The expert should have a certificate indicating such knowledge.

The contractor is responsible that the supervisor as well as the personnel responsible for manufacturing and testing the concrete are trained and instructed at suitable intervals, so that they are able to cary out all measures for the concrete work, including testing and quality control. The supervisor of a concrete laboratory should keep records on the training of the personnel.